VISCOELASTIC CHARACTERIZATION OF BLENDED BINDERS FOR ASPHALT PAVEMENT RECYCLING

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by

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This thesis is dedicatyed to my parents:

Esmat Ameri and Hassan Soleymani

and to my wife:

Fatemeh Saboni

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ABSTRACT

The main purpose of this dissertation was to study the SHRP Performance-Graded (PG) binder system for selection of recycling agents for asphalt pavement recycling projects. Traditional asphalt cement testing methods such as viscosity and penetration have several shortcomings and could not characterize the binders at a wide range of temperatures and loading times. The PG system characterizes asphalt cement with cyclic and creep tests in a wide range of temperatures and loading times. The PG system makes it possible to relate the testing parameters with the main asphalt pavement distresses such as fatigue, rutting, and low-temperatures cracking. SHRP did not look at the asphalt pavement recycling. Therefore, it was necessary to study the PG binder system for asphalt recycling.

One asphalt cement was selected as the original binder in the reclaimed asphalt pavement. Two soft asphalt cements and two recycling agents were selected as the rejuvenator. The original binder was aged in the laboratory and blended with rejuvenating materials. The resultant 10 blended binders were characterized with a Dynamic Shear Rheometer and a Bending Beam Rheometer apparatus. The testing procedure used involved performing sweep temperatures ranging from –30 to 70°C.

The PG testing results were used to develop some numerical models for characterization of blended binders. The proposed models are based on PG binder testing parameters such as complex shear modulus (G*), phase Angle (δ), stiffness (S), and m-value. The analysis showed that a linear relationship is accurate enough for prediction of PG binder parameters with proportion of recycling agents. The temperature dependency of blended lines for complex shear modulus was studied. The temperature dependencies of blended lines for phase angle and m-value were studied and a logarithmic model was selected for this purpose.

The loading time dependency of blended binders was studied by building the master curves. The SHRP A-002A binder model was used for estimating the rheological indexes (R) and crossover frequencies (ω_c) of blended binders. The regression analysis showed that a linear relationship is accurate enough for prediction of SHRP A-002A parameters with proportion of recycling agents of blended binders. The temperature dependency of shift factors was studied with defining temperature, T_d, from Williams, Landel, and Ferry (WLF) Equation.

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TABLE OF CONTENTS

PERMISSION TO USE i

ABSTRACT ii
ACKNOWLEDGEMENTS iv

TABLE OF CONTENTS v

LIST OF FIGURES viii

LIST OF TABLES ix

1. INTRODUCTION 1

1.1 GENERAL 1

1.2 IDENTIFICATION OF RESEARCH NEED 2

1.3 OBJECTIVES OF THE RESEARCH 3

1.4 SCOPE 4

1.5 THESIS ORGANIZATION 5

2 LITERATURE REVIEW 7

2.1 INTRODUCTION 7

2.2 IMPORTANCE OF ASPHALT CEMENT 8

2.3 NATURE OF THE ASPHALT CEMENT 8

2.3.1 Elemental Composition and Molecular Structure 9
2.3.2 Conceptual Composition Model 10
2.3.3 Colloidal System 10
2.3.4 Analytical Procedures 11

2.4 PHYSICAL PROPERTIES OF ASPHALT CEMENT 13

2.4.1 Aging of Asphalt Cement 13
2.4.2 Consistency Measurement Methods 16
2.4.3 Temperature Susceptibility Indices 18
2.4.4 Comparison Between Consistency Testing Results 22
2.4.5 Static Loading Tests 23

2.4.6 Dynamic Mechanical Test 25
2.4.7 Relationship of Creep and Dynamic Testing Parameters 27
2.5 ASPHALT CEMENT AS A VISCO-ELASTIC MATERIAL 30
2.5.1 Visio-Elastic Mechanical Model for Asphalt Cements 30
2.5.2 Time-Temperature Superposition and Master Curve 32
2.5.3 Mathematical Models for Asphalt Cements 36
2.5.3.1 Jongepier and Kuilman Model 36
2.5.3.2 Dobson Model 38
2.5.3.3 Dickinson and Witt Model 40
2.5.3.4 SHRP A-002A Model (Christensen and Anderson Model) 42
2.5.3.5 Other Studies 43
2.6 LOW-TEMPERATURE CHARACTERIZATION OF ASPHALT 44
2.7 ASPHALT GRADING SYSTEMS 47
2.8 CRITICAL REVIEW OF ASPHALT CEMENT SPECIFICATIONS 49
2.9 STRATEGIC HIGHWAY RESEARCH PROGRAM (SHRP) 50
2.9.1 Performance Graded (PG) Binder System 50
2.9.2 PG Binder Criteria for Pavement Distresses 53
2.9.3 The PG Binder Specifications 55
2.9.4 Critical Review of PG Binder System 56
2.10 RECYCLING AGENTS FOR ASPHALT PAVEMENT RECYCLING 58
2.10.1 Current Methods for Selection of Recycling Agent 59
2.10.2 Models for Blended Binders 62
2.11 CHAPTER SUMMARY 63

3. MATERIALS, TESTING EQUIPMENT, AND RESULTS 65

3.1 INTRODUCTION 65
3.2 MATERIALS 66
3.3 LABORATORY AGING 71
3.4 BLENDING PROGRAM 75
3.5 CONSISTENCY TESTING PROGRAM 77
3.6 PG TESTING PROGRAM 79

3.6.1 DYNAMIC SHEAR RHEOMETER (DSR).. 79
3.6.2 BENDING BEAM RHEOMETER (BBR).. 81

3.7 PRECISION OF THE PG TESTING RESULTS 83

3.7.1 REPEATABILITY OF THE DSR TEST 86
3.7.2 REPEATABILITY OF THE BBR TEST 89

3.8 DISCUSSION OF THE RESULTS 92

3.9 PERFORMANCE GRADE OF BINDERS 102

3.10 CHAPTER SUMMARY 103

4. MODELS FOR BLENDED BINDERS 106

4.1 INTRODUCTION 106

4.2 MODELS FOR BLENDED BINDERS 107

4.2.1 Models for Shear Modulus and Stiffness of Blended Binders.. 111
4.2.2 Temperature Dependency of Blended Lines for G* and S 114
4.2.3 Models for Phase Angle and m-value of Blended Binders.. 124

4.3 PERFORMANCE PREDICTION OF BLENDED BINDER 139

4.3.1 RUTTING 139
4.3.2 Fatigue 142
4..3.3 Low-Temperature Cracking.. 145

4.4 PROCEDURES FOR SELECTION OF RECYCLING AGENT 145

4.4.1 PG Criteria Procedure.. 146
4.4.2 PG Temperature Procedure 148
4.4.3 A Case Study.. 150

4.5 DISCUSSION.. 154

4.6 SUMMARY.. 154

5. MASTER CURVES FOR BLENDED BINDERS.. 157

5.1 INTRODUCTION 157

5.2 BUILDING MASTER CURVES 158

5.3 TEMPERATURE DEPENDENCY OF SHIFT FACTOR 164

5.4 CHARACTERIZATION OF BINDERS WITH MASTER CURVE PARAMETERS 175

5.5 CHANGE IN R AND ω_C OF BLENDED BINDERS 186

5.6 SUMMARY 191

6. CONCLUSIONS AND RECOMMENDDATIONS....................195

6.1 SUMMARY 195

6.2 CONCLUSIONS 196

6.3 RECOMMENDATIONS 198

REFERENCES....................200

APPENDIX A....................PG TESTING RESULTS

APPENDIX B....................COMPUTER PROGRAM FOR CALCULATING SHIFT FACTORS OF BLENDED BINDERS

LIST OF TABLES

Table 2.1 Elemental Analysis of Four Representative Petroleum Asphalts.......................... 9

Table 2.2 The SHRP PG Binder System 57

Table 3.1 Physical Properties of the 150-200 Asphalt Cement (Original Binder)........... 67

Table 3.2 Physical Properties of 200-300 and 300-400 Asphalt Cements....................... 68

Table 3.3 Physical Properties of Cyclogen-L (Cyclogen)... 70

Table 3.4 Physical Properties of Shell-Flex 210 (Flexon)... 70

Table 3.5 Mass Losses for Asphalt Cements and Blended Binders................................ 74

Table 3.6 Proportion of Aged Asphalt Cement and Recycling Agents 76

Table 3.7 Consistency Test Results for Asphalt Cements and Blended Binders............. 77

Table 3.9 Estimated PG Temperature Variability from SUPERPAVE Test Variability 87

Table 3.10 Statistical Summary of Stiffness Repeatability for BBR 87

Table 3.11 Summary of Coefficients of Variation of Complex Shear Modulus and Phase Angle for 150-200 Asphalt Cement 88

Table 3.12 Coefficient of Variations (Percent) for BBR Parameters 90

Table 3.13 Comparison of Coefficients of Variation (Percent) for PG Binder Tests...... 92

Table 3.14 The PG Temperature Variability as Measured Within Imperial Oil Lab 103

Table 3.15 The Performance Graded (PG) for All Binders Used in This Study 104

Table 4.1 Summary of Regression Analysis for Complex Shear Modulus and Stiffness of Blended Binders 113

Table 4.2 Summary of Statistical Analysis of Parallelism of the Regression Lines for Complex Shear Modulus and Stiffness...123

Table 4.3 Shift Values for Binders Blended with Recycling Agents (RA) 125

Table 4.4 Summary of Regression Analysis of Linear and Non-Linear Models for Phase Angle and m-Value of Blended Binders 127

Table 4.5 Summary of Statistical Analysis for Temperature Dependency of the Phase Angle (Unaged and Aged) and m-value..135

Table 4.6 Estimated Parameters for Unaged Phase Angle Model ($\delta=\alpha+\beta \ln T$) of Blended Binders, Temperature (°K) 136
Table 4.7 Estimated Parameters for Aged Phase Angle Model ($\delta=\alpha+\beta \ln T$) of Blended Binders, Temperature (°K) 137
Table 4.8 Estimated Parameters for m-Value ($m=\alpha+\beta \ln T$) of Blended Binders, Temperature (°K) 138
Table 4.9 Maximum and Minimum Pavement Temperatures for Saskatoon
Table 4.10 Performance Grade (PG) Asphalt Binders for Saskatoon 153
Table 4.11 Criteria Testing Temperatures for PG Asphalt Binder for The Saskatoon Area 153
Table 5.1 Estimated Shear Modulus for Aged 150-200 Asphalt Cement at Different FrequencieS with the SHRP A-002A Model 160
Table 5.2 Calculated Shift Factor Values for Virgin Blended Binders 165
Table 5.3 Calculated Shift Factor Values for Aged Blended Binders 166
Table 5.4 Various Suggested Values for Parameters in the WLF Equation 170
Table 5.5 Calculated Defining Temperature (T_d) from WLF Equation for all binders ... 172
Table 5.6 Estimated Rheological Indices (R) for Asphalt Cements 177
Table 5.7 Estimated Crossover Frequencies (ωc) for Asphalt Cements 178
Table 5.8 Evaluation Criteria for Linear and Non-Linear Master Curve Parameters for Binders Blended 190

LIST OF FIGURES

Figure 2.1 Van der Poel Nomograph for Determining Asphalt Cement Stiffnes..............19

Figure 2.2 Typical Stress-Strain Relationship in a Creep Test for Asphalt Cement........25

Figure 2.3 Simple Mechanical Model for Asphalt Cement.......................32

Figure 2.4 Time-Temperature Superposition Method for Building Master Curve 35

Figure 2.5 Master Curve Parameters..35

Figure 2.6 Stress-Strain Responses of Asphalt Cement with DSR Test...................... ..52

Figure 2.7 The m-Value from BBR Test 52

Figure 2.8 The Viscosity Chart for Determining the Type /Amount of Recycling Agent. 61

Figure 3.1 Change in Viscosity with Temperature for 200-300 and 300-400 Asphalt Cements 69

Figure 3.2 The Rolling Thin Film Oven (RTFO) Apparatus Used in this Research....... 72

Figure 3.3 The Pressure Aging Vessel Used in this Research..................................... 73

Figure 3.4 The Dynamic Shear Rheometer Apparatus Used in This Research.............. 80

Figure 3.5 Typical Output of DSR Test 82

Figure 3.6 The Bending Beam Rheometer Used in This Research 84

Figure 3.7 Typical Output of BBR test 85

Figure 3.8 Isochronal Curves of the Complex Modulus for Aged and Unaged Asphalt Cement (150-200, 200-300, and 300-400) 94

Figure 3.9 Isochronal Curves of the Complex Modulus for Aged and Unaged Binders Blended with 200-300 and 300-400 Asphalt Cements 95

Figure 3.10 Isochronal Curves of the Complex Modulus for Aged and Unaged Binders Blended with Cyclogen.. 96

Figure 3.11 Isochronal Curves of the Complex Modulus for Aged and Unaged Binders Blended with Flexon 97

Figure 3.12 Isochronal Curves of the Phase Angle for Aged and Unaged Asphalt Cements (150-200, 200-300, and 300-400)... 98

Figure 3.12 Isochronal Curves of the Phase Angle for Aged and Unaged Asphalt Cements (150-200, 200-300, and 300-400) 98
Figure 3.13 and 3.14 Isochronal Curves of the Phase Angle for Aged and Unaged Binders Blended with 200-300 and 300-400 Asphalt Cements 99
Figure 3.15 Isochronal Curves of the Angle for Aged and Unaged Binders Blended with Cyclogen 100
Figure 3.16 Isochronal Curves of the Phase Angle for Aged and Unaged Binders Blended with Flexon 101

Figure 4.1 Change in Complex Shear Modulus with Proportion of Recycling Agent for Unaged Binders Blended with 200-300 Asphalt Cemnet at Intermediate and High Pavement Temperatures 115
Figure 4.2 Change in Complex Shear Modulus with Proportion of Recycling Agent for Aged Binders Blended with 200-300 Asphalt Cemnet at Intermediate Pavement Temperatures 115
Figure 4.3 Change in Complex Shear Modulus with Proportion of Recycling Agent for Unaged Binders Blended with 300-400 Asphalt Cemnet at Intermediate and High Pavement Temperatures 116
Figure 4.4 Change in Complex Shear Modulus with Proportion of Recycling Agent for Aged Binders Blended with 300-400 Asphalt Cemnet at Intermediate Pavement Temperatures 116
Figure 4.5 Change in Complex Shear Modulus with Proportion of Recycling Agent for Unaged Binders Blended with Cyclogen at Intermediate and High Pavement Temperatures 117
Figure 4.6 Change in Complex Shear Modulus with Proportion of Recycling Agent for Aged Binders Blended with Cyclogen at Intermediate Pavement Temperatures 117
Figure 4.7 Change in Complex Shear Modulus with Proportion of Recycling Agent for Unaged Binders Blended Flexon at Intermediate and High Pavement Temperatures 118

Figure 4.8 Change in Complex Shear Modulus with Proportion of Recycling Agent for Aged Binders Blended with Flexon at Intermediate Pavement Temperatures 118
Figure 4.9 Change in Stiffness with Proportion of Recycling Agent for Aged Binders Blended with 200-300 Asphalt Cement at Low Pavement Temperatures ..119
Figure 4.10 Change in Stiffness with Proportion of Recycling Agent for Aged Binders Blended with 300-400 Asphalt Cement at Low Pavement Temperatures ..119
Figure 4.11 Change in Stiffness with Proportion of Recycling Agent for Aged Binders Blended with Cyclogen at Low Pavement Temperatures 120
Figure 4.12 Change in Stiffness with Proportion of Recycling Agent for Aged Binders Blended with Flexon at Low Pavement Temperatures 120
Figure 4.13 Change in Phase Angle with Proportion of Recycling Agent for Unaged Binders Blended with 200-300 Asphalt Cement at High and Intermediate Pavement Temperatures....129
Figure 4.14 Change in Phase Angle with Proportion of Recycling Agent for Aged Binders Blended with 200-300 Asphalt Cement at Intermediate Pavement Temperatures....129
Figure 4.15 Change in Phase Angle with Proportion of Recycling Agent for Unaged Binders Blended with 300-400 Asphalt Cement at High and Intermediate Pavement Temperatures....130
Figure 4.16 Change in Phase Angle with Proportion of Recycling Agent for Aged Binders Blended with 300-400 Asphalt Cement at Intermediate Pavement Temperatures....130
Figure 4.17 Change in Phase Angle with Proportion of Recycling Agent for Unaged Binders Blended with Cyclogen at High and Intermediate Pavement Temperatures....131
Figure 4.18 Change in Phase Angle with Proportion of Recycling Agent for Aged Binders Blended with Cyclogen at Intermediate Pavement Temperatures....131
Figure 4.19 Change in Phase Angle with Proportion of Recycling for Unaged Binders Blended with Flexon at High and Intermediate Pavement Temperatures...132

Figure 4.20 Change in Phase Angle with Proportion of Recycling Agent for Aged Binders Blended with Flexon at Intermediate Pavement Temperatures 132

Figure 4.21 Change in *m-value* with Proportion of Recycling Agent for Binders Blended with 200-300 Asphalt Cement at Low Pavement Temperatures 133

Figure 4.22 Change in *m-value* with Proportion of Recycling Agent for Binders Blended with 300-400 Asphalt Cement at Low Pavement Temperatures 133

Figure 4.23 Change in *m-value* with Proportion of Recycling Agent for Binders Blended with Cyclogen at Low Pavement Temperatures 134

Figure 4.24 Change in *m-value* with Proportion of Recycling Agent for Binders Blended with Flexon at Low Pavement Temperatures 134

Figure 4.25 Change in the Rutting PG Criterion (G*/sin δ) with Proportion of Recycling Agents for Binders Blended with 200-300 Asphalt Cement at High Pavement Temperatures 140

Figure 4.26 Change in the Rutting PG Criterion (G*/sin δ) with Proportion of Recycling Agents for Binders Blended with 300-400 Asphalt Cement at High Pavement Temperatures 140

Figure 4.27 Change in the Rutting PG Criterion (G*/sin δ) with proportion of Recycling Agents for Binders Blended with Cyclogen at High Pavement Temperatures 141

Figure 4.28 Change in the Rutting PG Criterion (G*/sin δ) with Proportion of Recycling Agents for Binders Blended with Flexon at High Pavement Temperatures 141

Figure 4.29 Change in the Fatigue PG Criterion (G*.sin δ) with Proportion of 143
Recycling Agents for Binders Blended with 200-300 Asphalt Cement at Intermediate Pavement Temperatures 143

Figure 4.30 Change in the Rutting PG Criterion (G*.sin δ) with Proportion of Recycling Agents for Binders Blended with 300-400 Asphalt Cements at Intermediate Pavement Temperatures 143

Figure 4.31 Change in the Rutting PG Criterion (G*.sin δ) with Proportion of Recycling Agents for Binders Blended with Cyclogen at Intermediate Pavement Temperatures....144

Figure 4.32 Change in the Rutting PG Criterion (G*.sin δ) with Proportion of Recycling Agents for Binders Blended with Flexon at Intermediate Pavement Temperatures....144

Figure 4.33 Charts for Selection of Recycling Agents (200-300 Asphalt Cement) Based on the PG Performance Criteria....147

Figure 4.34 PG Temperature Criteria Chart for Selection of Recycling Agent (300-400 Asphalt Cement)....151

Figure 5.1 Change in Complex Shear Modulus of Aged 150-200 Asphalt Cement Versus Frequencies at High and Intermediate Pavement Temperatures....161

Figure 5.2 Illustration of Time–Temperature Superposition Method for Aged 150-200 Asphalt Cement....162

Figure 5.3 Effect of Temperatures on Shift Factors for Unaged Blended Binder....167

Figure 5.4 Effect of Temperatures on Shift Factors for Aged Blended Binders....167

Figure 5.5, 5.6, 5.7, and 5.8 Effect of Temperatures on Shift Factors of Virgin and Aged Binders Blended With 200-300 and 300-400 Asphalt Cement....168

Figures 5.9, 5.10, 5.11, 5.12, 5.13, and 5.14 Effect of Temperatures on Shift factors for Virgin and Aged Binders Blended with Cyclogen and FleON....169

Figure 5.15 Change in T_d with Proportion of Recycling Agent (200-300 Asphalt Cement) in the Blends....173

Figure 5.16 Change in T_d with Proportion of Recycling Agent (300-400 Asphalt Cement) in the Blends....173

Figure 5.17 Change in T_d with Proportion of Recycling Agent (200-300 Asphalt Cement) in the Blends....174

Figure 5.18 Change in T_d with Proportion of Recycling Agent (200-300 Asphalt Cement) in the Blends....174

Figure 5.19 Master Curves for Unaged and Aged 150-200 Asphalt Cement 179
Figure 5.20 Master Curves for Unaged and Aged 200-300 Asphalt Cement 180
Figure 5.21 Master Curves for Unaged and Aged 300-400 Asphalt Cement 180
Figure 5.22 Master Curves for Unaged and Aged M1 Blended Binder 180
Figure 5.23 Master Curves for Unaged and Aged M2 Blended Binder 181
Figure 5.24 Master Curves for Unaged and Aged N1 Blended Binder 181
Figure 5.25 Master Curves for Unaged and Aged N2 Blended Binder 182
Figure 5.26 Master Curves for Unaged and Aged C1 Blended Binder 182
Figure 5.27 Master Curves for Unaged and Aged C2 Blended Binder 183
Figure 5.28 Master Curves for Unaged and Aged C3 Blended Binder 183
Figure 5.29 Master Curves for Unaged and Aged F1 Blended Binder 184
Figure 5.30 Master Curves for Unaged and Aged F2 Blended Binder 184
Figure 5.31 Master Curves for Unaged and Aged F3 Blended Binder 185
Figure 5.32 Master Curves for Binders Blended with 200-300 and 300-400 Asphalt Cement 187
Figure 5.33 Master Curves for Binders Blended with Cyclogen 188
Figure 5.34 Master Curve for Aged Binders Blended with Flexon 189
Figures 5.35, 36, 37, and 38 Change in Rheological Index (R) with proportion of Recycling Agent (% mass) in the Blend 192
Figures 5.35, 36, 37, 38 Change in Crossover Frequency (ω_c) with Proportion of Recycling Agent (% mass) in the Blends 193

CHAPTER 1

INTRODUCTION

1.1 GENERAL

There are more than 4 million kilometers of paved roads in North America. More than 93 percent of these roads are asphalt roads. The Road Information Program (TRIP, 1994) reported that more than 55% of the roads in North America are in poor to fair condition and require major maintenance or rehabilitation. The annual road rehabilitation and maintenance expenditures in North America are more than 20 billion dollars (Haas et al. 1994). Therefore, improvements in the design and construction of maintenance and rehabilitation methods will result in significant savings.

Asphalt pavement recycling is an effective technique for the rehabilitation of the asphalt pavements and has been implemented throughout North America. The Ontario Ministry of Natural Resources (MNR, 1992) reported that 534,000 tonnes of Reclaimed Asphalt Pavement (RAP) were used and an additional 788,000 tonnes stockpiled for subsequent recycling in 1990 in the Greater Toronto Area. The Federal Highway Administration (FHWA) estimates that 50 million tonnes of RAP is reused in the United States each year. This amount of RAP saves approximately 2800 million liters of asphalt binder and 47 million tonnes of aggregate. The value of this material conservation exceeds 1 billion dollars in direct saving per year.

The mixture design of recycled asphalt is more critical because of more variables involved, compared to the design of normal asphalt mixtures. The inherent savings of pavement recycling could be quickly outweighed if the proper selection of recycling agents, new aggregates and volumetric properties is not incorporated in recycled projects.

1.2 IDENTIFICATION OF THE RESEARCH NEED

The current methods for characterization of asphalt cements are based on testing procedures such as viscosity and penetration. These methods have several shortcomings. The penetration and viscosity characterize asphalt cement at intermediate and high service temperatures. In these traditional binder-testing methods, characterization at low temperatures is based on the extrapolation of the high and intermediate testing temperatures, which is not an accurate method. The other shortcoming in the current characterization methods of asphalt cement is that the effects of aging are not addressed adequately.

The Strategic Highway Research Program (SHRP) introduced the new Performance-Graded (PG) system for asphalt cement to address the shortcomings in current grading and specification systems. The PG system considered asphalt cement as a linear viscoelastic material which properties change with temperatures and loading times (or loading frequencies). The PG binder system uses cyclic, creep, and tensile loading tests to characterize and grade asphalt cements. In this new grading asphalt system two methods, Rolling Thin Film Oven (RTFO) and Pressure Aging Vessel (PAV), are used for simulating aging of asphalt cements in the laboratory. Although there are some debates regarding PG performance criteria, but it is the first asphalt cement specification that has tried to provide a direct relationship between binder testing and pavement performance.

The PG binder system was based on research conducted on normal asphalt cements and mixtures. Implementations of the PG system to recycled mixtures and polymer modified asphalt cements were not considered in the SHRP study. Many highway agencies in Canada and in the United States have implemented the PG binder system for the selection of asphalt cement. In order to extend the use of the PG binder system in recycled pavement, it is necessary to investigate the effect of other variables involving in the asphalt pavement recycling.

1.3 OBJECTIVES OF THE RESEARCH

The first purpose of this dissertation is to investigate the suitability of the PG binder system for asphalt pavement recycling. The main hypothesis is that *a linear relationship can predict PG parameters, complex shear modulus (G^*), phase angle (δ), stiffness (S), and m-value, of blended binders with change in proportion of recycling agents in the blends.* If a linear relationship exists then it can be used easily for selection of the type and amount of recycling agent in the design of recycled mixtures. This hypothesis was selected based on the observation of the current linear relationship between viscosity and proportion of recycling agent in the blended binders. The above hypothesis should be tested over a wide range of temperatures and aging conditions. Therefore, appropriate experimental design, analytical, and statistical methods are used to determine whether the hypothesis stated above is true.

The second objective of this research is to investigate the temperature and loading time dependencies of blended binders with their master curves. Master curves are a powerful tool for characterization of temperature and loading time dependencies of asphalt cements and blended binders. The relationships between master curve parameters, rheological index (R) and crossover frequency (ω_c), of blended binder with proportion of recycling agent are studied. These parameters (R and ω_c) are based on acceptance of the SHRP A-002A model for asphalt cement, which suggests a hyperbolic equation for master curve of asphalt cements, as subsequently described.

1.4 SCOPE

This study is a binder study, which considered only one original binder and four recycling agents including two soft asphalts and two oil-based materials. These are typical original binder and recycling agents that are used in cold climate conditions like in Canada in asphalt pavements. This research characterized 10 different blended binders and three asphalt cements with the PG testing equipment. No chemical asphalt cement tests was used in this study.

Instead of extracting old asphalt cement from RAP, laboratory aging of asphalt cement, which includes the Rolling Thin Film Oven (RTFO) and Pressure Aging Vessel (PAV), were used. The laboratory-aged binder is more homogeneous and its preparation takes less time compared to the extraction and recovery of aged binder from RAP. Although some studies have shown a good correlation between PG laboratory and field aged asphalt cement, a general conclusion need more validations for various materials and climate conditions.

Dynamic Shear rheometer (DSR), a cyclic loading test apparatus, and Bending Beam Rheometer (BBR), a creep loading test apparatus, were used for the characterization of asphalt cements and blended binders based on the PG specification. Because of unavailability of the direct tension apparatus, no direct tension test was conducted in this research.

The blended binders were characterized at a wide range of temperatures from -30°C to 70°C. All blended binders were tested with DSR at 10 rad/s. loading frequency. For building master curves, the complex shear moduli were estimated with the SHRP A-002A model, as subsequently described. The range of testing results covers the expected shear modulus and stiffness that may be used for selection of asphalt cements by highway agencies in Canada and the United States.

Statistical methods including linear and non-linear regression analysis and testing hypothesis were used to analyze the test data. The SPSS, release 6.1.2, computer package was used for non-linear regression analysis. A FORTRAN computer program was written to calculate shift factors and to build the master curves. The developed models could be used in the design of recycled asphalt mixtures. These models provide a performance–based method in the selection of recycling agent for asphalt pavement recycling.

There are some limitations in this research. Developed models are based on one laboratory aged asphalt cement and four recycling agents. The developed models and conclusions from this asphalt binder study must be verified with additional studies in this area. For better performance prediction of recycled pavements, it is necessary to investigate the effect of new variables, such as volumetric mixture parameters and the proportion of RAP in recycled mixtures.

1.5 THESIS ORGANIZATION

This thesis is organized into six chapters. The background and purposes of the research program are presented in this first chapter. The scope of the research program along with the objectives is discussed.

The second chapter provides a brief summary regarding composition and chemistry of asphalt cements. The traditional asphalt cement grading systems and their limitations are discussed. Because of the importance of asphalt cement at low pavement temperatures, a short review for the characterization of asphalt cements at low temperatures is presented. Different viscoelastic characterization methodologies (mechanical models, master curve and mathematical models) of asphalt cement have been reviewed. The new SHRP PG binder system, including testing and grading, is discussed. Finally, the current methods for selection of recycling agents and available models are reviewed.

The third chapter explains the physical properties of materials used in this study.

Laboratory aging and preparation of blended binders are described. The PG binder testing equipment including Dynamic Shear Rheometer (DSR) and Bending Beam Rheometer (BBR) are explained. Two methods for laboratory aging of asphalt cement including Rolling Thin Film Oven (RTFO) and Pressure Aging Vessel (PAV) are described. A small-scale precision study was conducted to determine the accuracy of test results.

The fourth chapter explains the methodology and results for developing models for blended binders based on the PG binder system. The linear and non-linear models for characterizing the PG parameters with change in proportion of recycling agent in the blends are compared. The performances of blended binders were predicted based on the PG performance criteria. Two procedures for the selection of recycling agent are reviewed. These methods are based on the main hypothesis identified in objectives of this research. In a case study, different blended binders were compare, based on PG binder criteria, for a Saskatchewan recycling project that typifies climate condition in Canada.

The fifth chapter includes some analytical discussion regarding building the master curves and the prediction of SHRP A-002A hyperbolic parameters for blended binders. The master curve can be used for characterization of temperature and loading time dependencies of visco-elastic materials such as asphalt cements. A computer program, which is used for calculating the shift factors and building master curves, is explained. The relationships between master curve parameters (R, ω_c) and proportion of recycling agents are studied. The temperature dependency of blended binders was studied with the defining temperature (T_d) from Williams, Landel, and Ferry (WLF) Equation.

Finally, the last chapter presents the summary and conclusions of the research program. Recommendations for further research work are also made based on the results of this study.

CHAPTER TWO

LITERATURE REVIEW

2.1 INTRODUCTION

Recycling is an environmentally sound and economically attractive method of rehabilitating asphalt pavements. To restore the aged asphalt cement properties in the old pavement, it is necessary to add recycling agent(s) to the reclaimed mixture. Selection of a proper recycling agent is a critical step in asphalt pavement recycling mixture design.

The purpose of this chapter is to review the literature covering asphalt cement characterization. As background, a short review of the chemical structure and composition of asphalt cement is included. A review of the literature regarding low temperature characterization and performance of the asphalt cement is presented. The shortcomings of the current methods for the characterization of asphalt cements and the Strategic Highway Research Program (SHRP) Performance Graded (PG) binder system are discussed. A review of literature on the characterization of the blended binder (the mixture of aged asphalt cement and recycling agents) and the current methods for selection of recycling agents for asphalt recycling are presented.

2.2 IMPORTANCE OF ASPHALT CEMENT

Asphalt cement is the most widely used binder in the pavement industry. The asphalt cement binds the aggregate particles together enhancing the stability of the mixture and providing resistance to deformation under induced tensile, compressive and shear stresses. The performance of the asphalt mixture is a function of the asphalt cement, aggregate and its volumetric properties. Asphalt cement is the main component, which controls the viscoelastic properties during production in the plant and service on the road.

Low-temperature cracks are the main distress of asphalt pavements in cold areas. Highway agencies in Canada and northern United States are spending considerable sums for maintenance of low-temperature cracks on their asphalt pavements. Low-temperature cracks not only decrease the serviceability of the pavement for the road users but also permit the penetration of water to the pavement layers, which eventually causes other major pavement distresses. Many researchers believe that asphalt cements are the dominant controlling variable in low-temperature cracking distress mechanism (Culley 1969; Fromm and Phang 1971; Haas 1973; Fabb 1974; Kandhal and Koehler 1987). Therefore, characterization and proper selection of asphalt cement is an essential step in the asphalt mix design.

2.3 NATURE OF THE ASPHALT CEMENT

The main emphasis in this dissertation is on the physical properties of asphalt cement. In order to understand asphalt cement behavior, understanding of the chemical structure and composition of asphalt cement is essential.

Attempts have been made to correlate the chemical properties of asphalt cement to its laboratory physical properties and field performances. Garick and Wood (1986), Brule' et al. (1986), and Bishara and McReynolds (1990) reported that asphalts having similar physical properties could have different chemical compositions. Goodrich et al. (1986) studied the relationship between chemical composition and physical properties (such as penetration and viscosity) of some asphalt cements and concluded that although some asphalt cements had a similar composition, their physical properties were very different.

Researchers used four different approaches to study the chemical structure of asphalt cement:

1. Elemental composition and molecular structure
2. Conceptual composition model
3. Colloidal system
4. Analytical procedures.

2.3.1 Elemental Composition and Molecular Structure

Asphalt cement is a hydrocarbon that is composed mostly of carbon and hydrogen molecules. By weight, asphalt cement is typically 85 percent carbon, 10 percent hydrogen, and the balance, which is called heteroatom content, consists of sulfur, oxygen, nitrogen, and various metals such as vanadium and iron. Table 1 shows an elemental analysis of four representative petroleum asphalts.

Table 2.1 Elemental Analysis of Four Representative Petroleum Asphalts
(Peterson, 1984)

Asphalt Cement	**A**	**B**	**C**	**D**
Carbon , percent	83.77	85.78	82.90	86.77
Hydrogen, percent	9.91	10.19	10.45	10.93
Nitrogen, percent	0.28	0.26	0.78	1.10
Sulfur, percent	5.25	3.41	5.43	0.99
Oxygen, percent	0.77	0.36	0.29	0.20
Vanadium, ppm	180	7	1380	4
Nickel, ppm	22	0.4	109	6

Heteroatoms play an important role in the physical properties of asphalt. The polar heteroatom-containing compounds are capable of intermolecular associations, which affect such physical properties as boiling point, solubility, and viscosity. These polar compounds tend to be concentrated in the asphalt fraction of a crude oil (Roberts et al. 1991).

2.3.2 Conceptual Composition Model

According to the most simple and generally accepted concept of asphalt composition, asphalt cements are considered to made up of asphaltenes, resins, and oils. Asphaltenes are generally dark brown fabric solids and are defined as the n-pentane insoluble fraction of asphalt cements. The amounts and characteristics of asphaltenes vary considerably from asphalt to asphalt. Resins and oils form the maltene phase of the asphalt cement, which is soluble in n-pentane. Resins are generally dark and semi-solid or solid in character. They are fluid when heated and become brittle when cold. Resins work as agents that disperse (or "peptize") the asphaltenes throughout the oils to provide a homogeneous liquid. During oxidation, resins yield asphaltene type molecules. Oils are usually colorless or white liquids. They have paraffinic and naphthenic structures with no oxygen but nitrogen usually present. During oxidation, oils yield asphaltene and resin molecules (Roberts et al. 1991).

2.3.3 Colloidal System

Asphalt cement is not a true solution but is considered a colloidal or micellar system. Peterson (1984) showed that the compatibility and relationships of different components in the microscopically homogeneous mixture controls the overall behavior of asphalt cement rather than the quantitative amount of any single component. Nellensteyn (1928) first recognized its colloidal nature by stating that it has a dispersion of micelles in an oily medium. The relative amounts and characteristics of asphaltenes, resins, and oils present in asphalt cement influence its physical properties and performance in the Hot Mixture Asphalt (HMA). These influences make the asphalt act as a "sol," "sol-gel," or "gel". "Sol" asphalt binders typify a system in which the resin keeps the asphaltenes highly "peptized" (or dispersed) in the oily phase. "Sol" asphalts largely exhibit Newtonian flow characteristics. "Gel" asphalt binders typify a system in which resins are not very effective in peptizing asphaltenes. An excessive presence of paraffins in relation to the nitrogen bases also tends to reduce solubility, leading to increased "gel" characteristics, and suggesting increased separation of the dispersed and dispersing phases. "Gel" asphalt binders exhibit largely non-Newtonian behavior. "Sol-gel" is an intermediate between "sol" and "gel" (Roberts et al. 1991).

2.3.4 Analytical Procedures

The two most frequently used fractionation methods for asphalt cement are Rostler and Sternberg's chemical precipitation method and Corbett's selective adsorption-desorption (chromatographic) method. Although these methods can give some ideas regarding the "generic" fractions of asphalt binders, asphalts with the same generic fractions may have quantitatively different physical properties.

The adsorption-desorption method, developed by Corbett (1969) and standardized by ASTM D 4124, separates and quantifies the components of asphalt binders into four components: Asphaltenes (A), Saturates (S), Naphthene Aromatic (NA), and Polar Aromatic (PA). Rostler and Sternberg (1949) attempted to identify and quantify five components in asphalt cement. Five components that can be separated in asphalt cements are asphaltenes (A), nitrogen bases (N), first and second acidaffins (A1, A2) and paraffins (P). Rostler (1965) defined the *compatibility ratio* and *durability parameter* based on the amount of the fractions for asphalt binders as:

$$\text{Compatibility Ratio} = N/P \qquad [2.1]$$

$$\text{Durability Parameter or Rostler Parameter} = (N+A1)/(P+A2) \qquad [2.2]$$

Gotolski et al. (1968) defined another parameter, known as the *Gotolski parameter*, based on the same component parameters as shown in [2.3]:

Gotolski Parameter =(A1+A2+N)/(A+P) [2.3]

Anderson and Dukatz (1980) showed that *durability parameters*, for more than 400 asphalt cements used during 1950-1970, were most closely associated with their temperature susceptibility and that *Gotolski parameters* of these asphalts were more closely associated with their aging effects.

High Pressure Liquid Chromatography (HPLC) or High Pressure Gel Permeation Chromatography (HP-GPC) has been used to determine the Molecular Size Distribution (MSD). Jennings (1980) has classified asphalt binder molecules into Large Molecular Size (LMS), Medium Molecular Size (MMS), and Small Molecular Size (SMS). Jennings (1980) compared the relative amounts of LMS, MMS, and SMS in asphalt cements recovered from poorly performing asphalt pavements with those from good asphalt pavements. Results obtained from Montana roads indicate that greater relative amounts of LMS are associated with poor performance. The results of Kim et al. (1993) from studying three virgin asphalt cements and four recovered asphalt cements indicated that higher relative quantity of LMS caused lower tensile strength and resilient modulus of asphalt concrete and consequently poorer performance.

Bishara et al. (1991) showed the MSD in 11 asphalt cements, obtained by using the Corbett method (ASTM D4124, Method B), significantly correlate with some physical parameters such as Penetration Viscosity Number (PVN), viscosity at 135°C, and Viscosity Temperature Susceptibility (VTS). There was no significant correlation between MSD and some other physical parameters such as penetration at 25°C and viscosity at 60°C.

Recent work in the Strategic Highway Research Program (SHRP A-368) showed Size Exclusion Chromatography (SEC) has the potential to characterize the strongly associated molecular components. The molecular components play a major role in determining the rheological properties and aging characteristics of asphalt cement.

In summary, although chemical analysis of asphalt cement has unquestionable value in understanding the physical properties and asphalt pavement performance, physical and rheological tests have been shown to correlate better to pavement performance (Goodrich et al. 1986). Therefore, physical property measurements continue to be a primary means of specifying and selecting asphalt cement.

2.4 PHYSICAL PROPERTIES OF ASPHALT CEMENT

Physical properties of asphalt cement are important for grading, selection, and performance prediction of asphalt mixtures and pavements. For asphalt cement to be used successfully in a pavement, it must have the following characteristics:

- be capable of being made sufficiently fluid, either by heat or by the addition of a volatile solvent, to be pumped or sprayed, and to coat and "wet" the mineral aggregates
- be, or become, so viscous at high pavement temperatures that the finished surfacing will resist deformation (rutting)
- be flexible at low pavement temperatures that the finished surfacing will resist fracture (low-temperature cracking) and disintegration.

Any characterization and classifying of asphalt cements must consider the change in physical properties because of passing time (aging) of asphalt cement.

2.4.1 Aging of Asphalt Cement

The most important change in asphalt binder during construction and its service in a pavement is hardening or aging. Aging of asphalt cement, resulting from the plant mix and lay down process (short term aging) and in-situ field aging (long term aging), is an extremely complex phenomenon due to numerous influencing factors. Aging was originally referred to as a change in the chemical properties of an asphalt binder. However, because of the difficulty in measuring chemical properties, asphalt cements have historically been "graded" by specifications based on ranges or groupings of consistency values at one or more temperatures.

Vallerga et al. (1957) and Finn et al. (1967) reported different mechanisms that contribute to the age hardening of asphalt cements during mixing and/or in service as below:

1. oxidation
2. volatilization
3. polymerization
4. thixotropy
5. syneresis
6. separation.

Oxidation is the reaction of oxygen with a binder. The rate of oxidation depends on the character of the asphalt cement and the temperature. Volatilization is the evaporation of lighter constituents from an asphalt binder and is primarily a function of temperature. It is usually not a significant factor contributing to long-term aging in the pavement. Polymerization is a combination of like molecules to form larger molecules, causing a progressive hardening. Thixotropy is a progressive hardening due to the formation of a structure within an asphalt binder over a period of time, which can be destroyed somewhat by reheating and working the material. Syneresis is an exudation reaction in which the thin oily liquids are exuded to the surface of the asphalt binder film. With the elimination of these oily constituents, the asphalt binder becomes harder. Separation is the removal of the oily constituents, resins, or asphaltenes from the asphalt binder as caused by selective absorption of some porous aggregates (Roberts et al. 1991).

Traxler (1963) suggested nine additional factors affecting aging. They included several effects of light, water, chemical reaction with the aggregate, microbiological deterioration, and adsorption of heavy asphalt components on the surface of the aggregates. According to studies by Brown et al. (1957), Lee (1973), and Kandhal and Koehler (1984), the changes in physical properties of asphalt binders, such as penetration and viscosity, follow a hyperbolic function, which approach a definite limit with time. According to this theory, the following equation expresses the age hardening of asphalt in the field:

$$\frac{T}{\Delta Y} = a + bT \qquad [2.4]$$

where:

ΔY= Change in test property (such as penetration and viscosity) with time T or the difference between the zero-life value and the value at any significant time

a and b = constants

The extent of age hardening can be quantified in terms of penetration (percent retained penetration) or viscosity (aging index) as follows:

$$\%\ \text{Retained Penetration} = \frac{\text{Penetration of aged asphalt}}{\text{Penetration of original asphalt}} \times 100 \qquad [2.5]$$

$$\text{Aging Index} = \frac{\text{Viscosity of aged asphalt}}{\text{Viscosity of original asphalt}} \qquad [2.6]$$

The Thin Film Oven (TFO) developed by the California Highway Department is one of the important efforts for considering the aging effect on asphalt cements. This method was later modified as the Rolling Thin Film Oven (RTFO) and has been standardized by ASTM D2872. During the early 1960s when the viscosity grading system was being developed, the California Department of Highways developed a parallel Aged Residue (AR) viscosity grading classification based on the viscosity of the aged residue resulting from the RTFO. Recently the SHRP PG binder system selected the RTFO and introduced the Pressure Aging Vessel (PAV) to simulate short and long term aging of asphalt cements in a laboratory.

The methods for characterization of the physical properties of asphalt cement can be categorized into three groups: consistency, static, and cyclic (dynamic) mechanical methods.

2.4.2 Consistency Measurement Methods

Consistency can be described as the degree of fluidity at any particular temperature. Since asphalt cement is a thermoplastic material, its consistency varies with the temperature. Therefore, it is necessary to measure the consistency of different asphalt cements at the same temperature and shear loading conditions if comparisons are to be made.

Although different methods for the characterization of asphalt cements exist, consistency methods have been used more widely than other methods. The most important consistency test methods are penetration, viscosity, softening point, ductility, and a combination of these methods. Some of the current consistency tests are empirical, meaning that pavement performance experience is required before the test results yield meaningful information. A short review of these methods is presented below.

Penetration is an empirical test used for the grading of asphalt cement. Usually penetration is measured at 25°C, which approximates the average service temperature of asphalt pavements. A needle with a 100-g weight is allowed to penetrate the asphalt binder sample for 5 seconds. The depth of penetration is measured in units of 0.1 mm (dmm) and is reported as penetration units (ASTM D5).

It is difficult to specify the exact meaning of penetration test, but Van der Poel (1954) showed that it is approximately equivalent to the measurement of "stiffness" at a loading time of 0.4 seconds. For a gel material with a high penetration index, this loading time corresponds to a comparatively rapid loading and will involve considerable non-

Newtonian effects. Thus, it is well known that "blown" asphalt cements, which are produced by blowing air through the topped crude fraction, will have a much higher viscosity than distilled asphalt cements of the same penetration. Further disadvantages of the penetration test may be seen from the experiment of a thin surface layer which will be considerably affected by any "skin" effects such as may be experienced in the presence of the wax in asphalt cement.

Viscosity is the ratio of shear stress to the shear rate at any given temperature. Asphalt viscosity, unlike the empirical tests of penetration and ductility, is a fundamental measure of the asphalt flowability that is not affected by changes in testing conditions, such as the configuration of test instruments or the geometry of the sample. Viscosity tests are performed at 60°C (Absolute Viscosity) and 135°C (Kinematic Viscosity) to provide information regarding the intermediate to high temperature viscous behavior of the asphalt cement.

Absolute viscosity is a fundamental measure of a Newtonian fluid whose properties are independent of rate of loading or stress level. Asphalt exhibits Newtonian behavior only at high temperatures or at very low shear rates. At lower temperatures, typical of pavement in-service temperature, or short loading times, typical of traffic loading, asphalts are not Newtonian and cannot be described by an absolute value of coefficient of viscosity. The viscosity test at 60°C commonly uses two different capillary tube viscometers. One is the Asphalt Institute and the other is the Cannon-Manning vacuum viscometers (ASTM D2171 and AASHTO T202).

Kinematic viscosity characterizes the flow behavior under gravity and is the ratio of the viscosity to the density of a liquid. A Cannon-Fenske or Zeifuchs cross-arm viscometer is used to measure the Kinematic viscosity based on ASTM D2170 and AASHTO T201. At this temperature, the asphalt cement is sufficiently fluid to flow through the capillary tube under gravitational force alone, and there is no need to apply any partial vacuum. This test temperature was selected because it approximates the mixing and laydown temperatures during the construction of asphalt pavements.

The Ring and Ball (R&B) method in accordance with ASTM D36 and AASHTO T53 measures softening point. It can be simply defined as the temperature at which asphalt cement cannot support the weight of a specified steel ball and starts flowing. Its purpose is to determine the temperature at which a phase change occurred in the asphalt cement. Although its use in specifying paving asphalt is quite common in Europe, it is mostly used in the United States for high viscosity roofing asphalts.

The ductility of a paving asphalt cement is measured by the distance to which it will elongate before breaking when two ends of a briquette specimen are pulled apart at a specific speed and temperature. ASTM D113 and AASHTO T51 give the test procedure to measure ductility at 25°C and lower temperatures. Ductility test results obtained at 4°C or 15.6°C indirectly reflect the relationship between viscosity and shear susceptibility at these service temperatures.

Some researchers studied the relationship between ductility and pavement performance and concluded that there was a high correlation between ductility and low-temperature cracking. Some other investigators debated the ductility test results because of its imperial nature and poor reproducibility (Abson and Burton 1964; Kandhal 1977; Kandhal and Koehler 1984).

2.4.3 Temperature Susceptibility Indices

Shortcomings in consistency testing methods at low-temperatures and poor correlation among these tests have led some researchers to enhance a combination of consistency methods for the characterization of asphalt cements.

Van der Poel (1954) used the penetration and softening point for estimating the stiffness of asphalt cements. Van der Poel Nomograrph, in Figure 2.1, has made a major

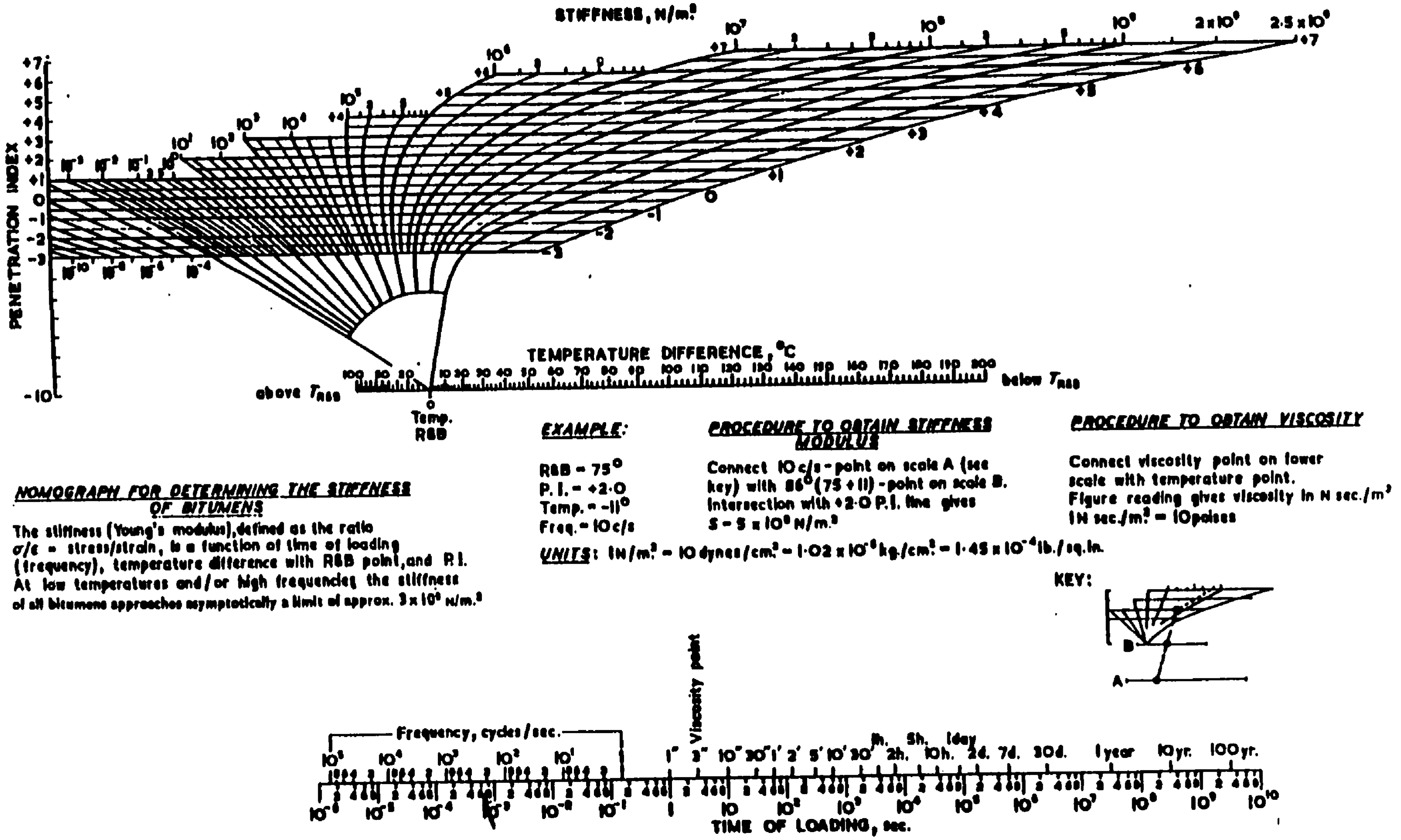


Figure 2.1 Van der Poel Nomograph for Determining Asphalt Cement Stiffnes
(from Van der Poel 1954)

contribution to understanding the rheology. The Van der Poel nomograph, determines the stiffness of asphalt cement at any temperature and loading time.

Some of the other important temperature susceptibility indices are Penetration Viscosity Number (PVN), Penetration Index (PI), and Viscosity Temperature Susceptibility (VTS).

McLeod (1976) proposed the use of PVN to determine the temperature susceptibility of asphalt cements. The PVN is calculated based on penetration at 25°C and viscosity at 135°C of the asphalt binder. The PVN value of paving asphalt can be calculated from the following equation:

$$PVN = -1.5\left(\frac{\log L - \log X}{\log L - \log M}\right) \qquad [2.7]$$

where:

X = viscosity at 135°C associated with the penetration at 25°C of the asphalt that the PVN value is required (centistoke).

L = for the same penetration at 25°C, the corresponding viscosity at 135°C for which the PVN = 0.0 or = 4.258-0.79674 log P (centistoke).

M = for the same penetration at 135°C, the corresponding viscosity at 135°C for which the PVN = -1.5 or = 3.46289-0.61094 log P (centistoke).

P = penetration at 25°C of the asphalt for which the PVN value is required.

Based on the study of four test pavements in Ontario and some other test pavements, McLeod (1978, 1987) concluded that the PVN of asphalt cement is associated with low temperature transverse pavement cracking. This was strongly supported by Haas et al. (AAPT 1988) where low-temperature cracking of 26 pavements across Canada was directly related to PVN of the extracted binders. McLeod (1989) extended the application of the PVN for the selection of asphalt cement in areas withoutfrost and for other applications such as asphalt recycling and surface treatments.

Pfeiffer (1950) expressed the temperature susceptibility quantitatively by a term designated as the *Penetration Index* (PI). Pfeiffer developed an equation for calculating the penetration index, using the penetration and ring-and-ball softening point temperature:

$$PI = [30/(1+90B)]-10 \qquad [2.8]$$

where:

$$B = [2.9031-\log(Pen25)]/(T_{R\&B}-77) \qquad [2.9]$$

where Pen 25 is the penetration at 25°C and $T_{R\&B}$ is the ring and ball softening point temperature, °F.

Because Pfeiffer PI does not work for waxy asphalts, Heukelom (1973) developed another method for calculating the penetration index based on the slope of the log penetration versus temperature plot. In this method the modified PI could be calculated from the following empirical equation:

$$PI = \frac{20 - 500A}{1+50A} \qquad [2.10]$$

where A is calculated from the equation below:

$$A = \frac{\log \text{ Pen at } T_1 - \log \text{ Pen at } T_2}{T_1 - T_2} \qquad [2.11]$$

where T_1 and T_2 are temperatures in degrees Celsius.

Values for the penetration indices calculated using these two methods do not necessarily agree, although the range in values is similar. The lower the PI value of asphalt cements the higher its temperature susceptibility. Most paving asphalt cements have a penetration index between +1 and -1. Asphalt cements with a PI below -2 are highly temperature susceptible, usually exhibit brittleness at low temperatures, and are very prone to transverse cracking in cold climates.

Another temperature susceptibility indexes is Viscosity Temperature Susceptibility (VTS). For determining VTS, a double logarithm of viscosity in centistoke is plotted against the logarithm of the absolute temperature in degrees K (empirical Walte's equation). Such plots generally result in straight lines, with the slope of the line equal to the VTS as follows:

$$\mathrm{VTS} = \frac{\log\ \log \text{viscosity at } T_2 - \log\ \log\ \text{viscosity at } T_1}{\log T_1\ -\ \log T_2} \qquad [2.12]$$

Larger VTS numbers indicate higher temperature susceptibility. Generally, the slope of lines which are measured at temperatures lower than 60°C tend to deviate from the slope established between 60°C and higher temperatures. This is because shear dependent viscosities are encountered below 60°C. The numerical differences between the VTS values of different asphalt cements are not large. The VTS values ranged from 3.36 to 3.98 for asphalts sampled in the United States.

2.4.4 Comparison Between Consistency Testing Results

The relationship between penetration and viscosity was studied by Saal and Labout (1936), Traxler and Pittman (1936), Mack (1939), Pendelton (1943), Van der Poel (1954), Welborn et al. (1966), Puzinauskas (1967, 1980), and Hoffman (1970). Some of these investigators have tried to develop a general relationship between viscosity and penetration. Some others showed that there is a poor correlation between viscosity and penetration or ductility values and that these relationships become poorer with decreasing temperature. This is due to unknown and variable shear rates associated with these tests at low temperatures. Attention must be given to stress levels or shear rates, or both, to attain a direct comparison between penetration and viscosity. In general, the penetration test involves higher stresses and a shorter loading time than do most viscosity tests.

In Special Report 91-5 of the U. S. Army Corps of Engineers (1991), Robertson stated there is very little correlation between the PI and the PVN values for a given asphalt cement. He pointed out the temperature susceptibility of asphalt cement is much better defined by its PI than its PVN. In the same report, Puzinauskas stated that when a comparison is made between PI and VTS, a poor correlation is obtained. A better correlation is found between PVN and VTS, and a poor correlation exists between PI and PVN.

2.4.5 Static Loading Tests

Static loading tests have been used for the characterization of time dependency of asphalt cements. In a static loading technique, three main loading modes can be used: creep, relaxation, and constant deformation modes. In creep mode, a constant load is applied while the deformation is measured with time. In relaxation mode, a constant deformation is applied while the load required keeping that deformation constant is measured with time. In a constant deformation mode, the material is exposed to a constant rate of deformation while the load required to keep that rate constant is measured. The moduli that are calculated from any of the three modes are all interrelated and describe the material behavior as a function of loading time.

The creep mode, as the simplest and most convenient method, has been mostly used for asphalt cements. Schweyer (1974) introduced different viscometers for studying the creep behavior of asphalt cements. These included a rotational type, which utilizes coaxial cylinder or cone and plate, and a specialized capillary type in which a piston is used to drive the asphalt through a capillary tube. Some other researchers used other types of viscometers or rheometers for creep test. These include the cone and plate viscometer developed by Asphalt Institute and standardized by ASTM D 3205, the sliding plate rheometer developed by the Bureau of Public Roads and others, the Shell sliding-plate rheometer developed by Fenijin and Krooshof (1970), and the bending beam rheometer developed by the Strategic Highway Research Program (Bahia et al. 1992).

Figure 2.2 shows a typical constant stress creep test result. Three different parts could be distinguished in a creep curve. A slight slope of the linear response depicts the initial elastic response and its recovery. The initial elastic response can be used to calculate the elastic modulus (E_0) or the shear modulus (G_1). The creep portion of the response curve eventually becomes linear, giving a constant slope or velocity. This slope is the strain or shear rate ($\dot{\varepsilon}$ or $\dot{\gamma}$) corresponding to a given stress and test temperature. Upon release of the applied stress, there is immediate recovery of elastic strain followed by the gradual, time-dependent recovery of elastic strain (delayed elastic strain). The residual strain that exists after complete elastic recovery is the nonrecoverable strain or permanent deformation, which essentially equals creep strain (Tia and Ruth 1987).

In analyzing the data from a creep test on asphalt cement, use is generally made of the stiffness moduli which is defined as:

$$S(t) = \sigma_0 / \varepsilon(t) \qquad [2.13]$$

where:

$S(t)$= time dependent modulus of stiffness

t = loading time (second)

σ_0 = applied uniaxial stress

$\varepsilon(t)$ = resulting uniaxial strain at time of t.

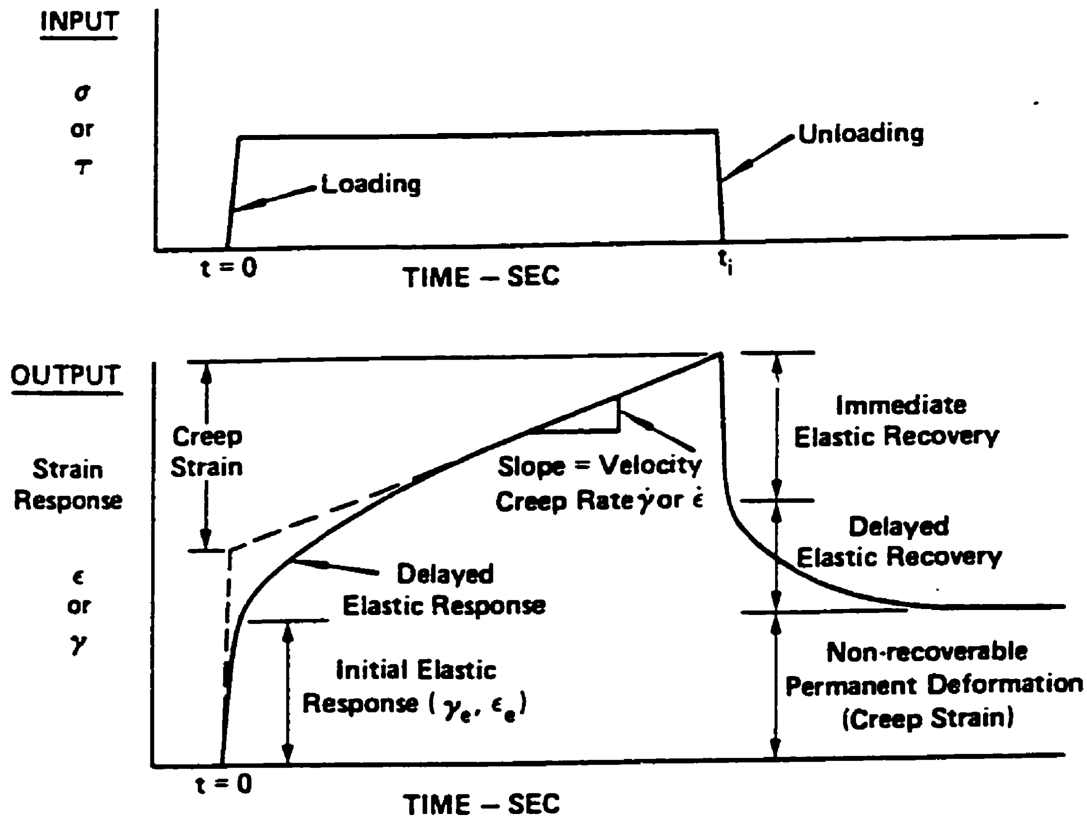


Figure 2.2 Typical Stress-Strain Relationship in a Creep Test for Asphalt Cement (from Tia M. and Ruth B. E. 1987)

2.4.6 Dynamic Mechanical Test

In dynamic (cyclic) mechanical analysis, a sinusoidal strain is applied to a specimen and the resulting stress is monitored as a function of frequency. This is termed *strain controlled* testing and is more common than *stress controlled* dynamic mechanical analysis, in which a sinusoidal varying stress is applied and the strain response is measured. If the behavior of the material is linear in controlled stress tests, the strain will also be sinusoidal but generally will be out of phase with the stress. The primary response in dynamic testing is the complex modulus (G*), which is computed in strain-controlled testing using the following equation:

$$G^*(\omega) = |\tau(\omega)| / |\gamma(\omega)| \quad [2.14]$$

where:

$G^*(\omega)$ = complex dynamic shear modulus at frequency ω, Pa

$|\tau(\omega)|$ = absolute magnitude of the dynamic shear stress response, Pa

$|\gamma(\omega)|$ = absolute magnitude of the applied dynamic shear strain, m/m.

The modulus may be resolved into two components, one in phase with the strain, $G'(\omega)$, and the other 90 degrees out of phase with the strain, $G''(\omega)$. This process is conveniently described by the following formula:

$$G^* = G' + iG'' \quad [2.15]$$

or

$$|G^*| = \left(G'^2 + G''^2\right)^{0.5} \quad [2.16]$$

where:

$G'(\omega)$ = dynamic storage modulus at frequency ω

$G''(\omega)$ = dynamic loss modulus at frequency ω

i = complex number = $\sqrt{-1}$

The other response of the dynamic mechanical test is the phase angle (δ). Phase angle represents the relative distribution of total response between an in-phase component and an out-of-phase component. These parameters can be related to each other through the following equations:

$$G'(\omega) = G^*(\omega) \cos \delta \quad [2.17]$$

$$G''(\omega) = G^*(\omega) \sin \delta \quad [2.18]$$

It is possible to derive the following equation from these relationships:

$$\tan\delta = G''/G' \qquad [2.19]$$

2.4.7 Relationship of Creep and Dynamic Testing Parameters

The dynamic mechanical properties of an asphalt cement can be directly related to its creep properties, in a mathematically complex way. Both characterizations give a complete indication of the viscoelastic properties of the tested material. Both the complex modulus and the stiffness modulus, in simple terms, are indicators of the resistance of asphalt cement to flow under a given set of loading conditions.

It is sometimes necessary to find a relationship between various rheological functions. As previously explained, there are three commonly used test methods for determining viscoelastic functions including creep, stress relaxation, and cyclic test.

These tests may be carried out using either shear or tensional/flexural loading. Each of these methods provides complete information on the linear visco-elastic behavior of the material, but in different forms. With the three common loading modes (creep, stress relaxation, cyclic) and the two general geometric load configuration (shear and tension/flexure), there are six commonly used viscoelastic functions:

- Creep compliance in shear, J(t)
- Creep compliance in tension/flexure, D(t)
- Relaxation modulus in shear, G(t)
- Relaxation modulus in tension or flexure, E(t)
- Dynamic complex modulus in shear, $G^*(\omega)$
- Dynamic complex modulus in tension or flexure, $E^*(\omega)$.

Various other viscoelastic functions can be defined through the complex modulus and the phase angle:

- Storage modulus in shear, $G' = G^* \cos \delta$
- Storage modulus in tension or flexure,, $E'' = E^* \cos \delta$
- Loss modulus in shear, $G'' = G^* \sin \delta$
- Loss modulus in tension or flexure, $E'' = E^* \sin \delta$
- Dynamic complex compliance in shear, $J^* = 1/G$
- Dynamic complex compliance in tension or flexure, $D^* = 1/E$
- Storage compliance in shear, $J' = \cos \delta / G^*$
- Storage compliance in tension or flexure, $D' = \cos \delta / E^*$
- Loss compliance in shear, $J'' = \sin \delta / G^*$
- Loss compliance in tension or flexure, $D'' = \sin \delta / G^*$

There are theoretically exact conversions among these functions, but the mathematics of these exact interrelations involve either equations or transformations (Laplace or Fourier) that in general can only be solved numerically. The flexural and tension moduli of asphalt cement could be assumed equivalent because the flexural test, such as the bending beam, can only be performed when the binder is in a rigid state and the strains are limited to one percent or less. Therefore, equivalent or nearly equivalent moduli in tension, compression, and flexure should be expected for asphalt cements (SHRP A-369).

The stiffness of the binder, as originally introduced by Van der Poel (1954), was defined as the inverse of the tension creep compliance. However, in asphalt technology, it is often defined by dynamic complex modulus in tension or flexure:

$$S(t) = 1/D(t) \approx E^*(\omega) \qquad [2.20]$$
$$t \rightarrow 1/\omega$$

The others interrelations for VEL functions that may be used are:

$$J(t) = 1/G^*(\omega) \qquad t \rightarrow 1/\omega \qquad [2.21]$$

$$D(t) = 1/E^*(\omega) \qquad t \rightarrow 1/\omega \qquad [2.22]$$

In converting shear and tension properties, a rigorous treatment requires the knowledge of the shear moduli and one other function, such as Poisson's ratio (μ) or the bulk modulus, K. Commonly treated in asphalt technology, incompressibility is assumed (μ = 0.5). The relationships between the shear and extensional relaxation moduli and compliance are then:

$$E(t) = 2(1+\mu)\,G(t) = 3\,G(t) \qquad [2.23]$$

$$D(t) = J(t)\,/\,[2\,(1+\mu)] = J(t)\,/\,3 \qquad [2.24]$$

Finally, the relationship between stiffness moduli, as defined by Van der Poel (1954) in terms of the shear compliance and dynamic modulus, could be as follows:

$$S(t) = 3\,/\,J(t) \qquad [2.25]$$

$$S(t) = 3\,G^*(\omega) \qquad t \rightarrow 1/\omega \qquad [2.26]$$

The results of the dynamic shear testing are most commonly given in terms of the complex modulus in shear (G*) and the phase angle (δ). The bending beam rheometer, on the other hand, produces data in the form of the flexural creep compliance, which is inverted to give a stiffness modulus. From the above discussions it should be apparent that the flexural stiffness at time (t) from the bending beam rheometer should be approximately equal to three times the dynamic shear modulus when the frequency is given in Hertz (SHRP A-369).

2.5 ASPHALT CEMENT AS A VISCO-ELASTIC MATERIAL

Asphalt cements are viscoelastic materials and their mechanical behavior is dependent on both the temperature and the duration of loading. At low temperatures and short loading times asphalt cements behave as elastic solids, while at high temperatures and long loading times they behave as simple viscous liquids. At intermediate temperatures and loading times, the behavior is more complex.

Jongepier (1969) showed that in addition to the temperature and loading time, a third factor can also influence the behavior of asphalt cement. This factor may show up as a dependence of the measured properties on stress, strain, or rate of strain, which is indicative of a change in or even the failure of the material during the test. These non-linear properties are probably related to failure characteristics of asphalt cement.

Broome and Bilmes (1941), Lethersich (1942), and Romberg and Traxler (1947) have studied nonlinear behavior of asphalt cement. It has been shown that two non-linear phenomena, thixotropy and stress dependence of viscosity, are observable in asphalt cements although these effects are not usually significant when the penetration index is low. The non-linear response, especially for viscous materials, is extremely difficult to characterize in the laboratory and to model in practical engineering applications. Fortunately, considering the linear behavior for characterization and analysis is generally more than adequate for engineering design problems. The Strategic Highway Research Program considered the asphalt cement as a linear viscoelastic material (SHRP A-369).

2.5.1 Visco-Elastic Mechanical Model for Asphalt Cements

The use of a visco-elastic mechanical model including spring and dashpot in series (Maxwell model), in parallel (Kelvin model), and the combined Maxwell-Kelvin

(Burger) arrangement have been proposed for asphalts. These may apply for "sol" type, highly aromatic-naphtenic materials, but for highly saturated naphtanic paraffin materials, the "gel" type, it is necessary to consider power law effects in the rheology. Kovacs (1961) considered a modified deformation-dependent Kelvin dashpot in the delayed elastic part of the model and omitted the lower Maxwell dashpot because these will not generally fit asphalt cement behavior.

Lee and Marwick (1937) suggested a dashpot and spring mechanical model for asphalt cement. Saal and Labout (1958) used a simple mechanical model such as in Figure 2.3 for asphalt behavior at short loading time, which is a combination of a Maxwell and a Voigt element. For this model, the following equation could have been obtained for deformation at constant stress:

$$\varepsilon = \frac{\sigma}{E_1} + \frac{\sigma}{Q_1} \cdot t + \frac{\sigma}{E_2} (1 - e^{-t/\lambda_r}) \qquad [2.27]$$

where:

$$\lambda_r = \frac{Q_2}{E_2} = \text{retardation time.}$$

Differences in rheological properties between asphalts can be represented in the model by different values for the parameters.

Brown et al. (1957) discussed some experimental data for asphalt cement using a Maxwell-multiple Kelvin model. Schweyer et al. (1977) presented another model. In this model, the lower (Maxwell) dashpot has been modified to incorporate into the system a self-generating feedback system that causes the dashpot to increase (or decrease) in cross-section areas.

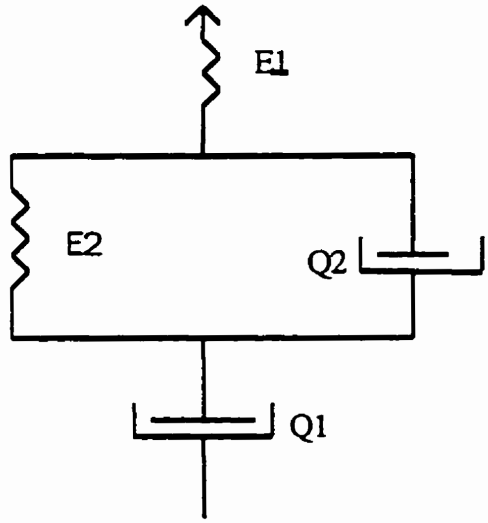


Figure 2.3 Simple Mechanical Model for Asphalt Cement (from Eirichi E. R. 1958)

2.5.2 Time-Temperature Superposition and Master Curve

Many researchers (Brodnyan 1958; Sisko and Brunstrum 1968; Dobson 1969; Duthie 1972; Pink et al. 1980; Macarroni 1987; Goodrich 1988) have used the time-temperature superposition method to build the master curve for characterization of asphalt cement. Because of the importance of this method and its application in this research, a review of the method is presented below:

Theoretical and experimental results indicate that for a certain class of material, such as asphalt cement, the effect due to time and temperature can be combined into a single parameter. This can be done through the concept of the *time-temperature superposition principle* which implies that the following relationship exist:

$$G(T, t) = G(T_0, \zeta) \qquad [2.28]$$

where:

t is the actual time of observation measured from first application of load

T is the temperature

ζ is the reduced time which is related to the real time t by a temperature shift factor $a_T(T)$

T_0 is the reference temperature.

The time-temperature superposition principle cited above states that the effect of temperature on the time dependent mechanical behavior is equivalent to a stretching (or shrinking) of the real time for temperature above (or below) the reference temperature. In other words, the behavior of materials at high temperature and high strain rate is similar to that at low temperature and low strain rate (Findley et al. 1976).

In the construction of a master curve, using time-temperature superposition or method of reduced variable, data is first collected over a range of temperatures and frequencies. A standard reference temperature must be selected. Generally, when analyzing viscoelastic data for asphalt cements, a reference temperature of 25°C is used. The data at all temperatures is shifted horizontally, with respect to time of loading, until the curves merge into a single smooth function. The shifting may be done based on any of the viscoelastic functions (G^*, δ). If time-temperature superposition is valid, the other viscoelastic functions will all form continuous functions after shifting. This means that the shift factors should be the same for all viscoelastic functions (Ferry 1980).

The amount of shifting required at each temperature to form the master curve is of special importance and is called the shift factor, $a(T)$. A plot of $\log a(T)$ versus temperature is generally prepared in conjunction with the master curve. This type of plot gives a visual picture of how the properties of a viscoelastic material are changing with temperature.

A schematic representing the basic process involved in constructing a master curve is shown in Figure 2.4. Figure 2.5 shows a typical master curve and its parameters including: glassy modului, G^*_g, steady-state viscosity, η_0, crossover frequency, ω_c, or crossover time, t_c, and rheological index, R. Because different asphalt cements can show different shapes of master curve, these parameters could be used for the characterization of asphalt cements. A short description of these parameters is presented below:

- **The glassy moduli, G^*_g**--the value that complex modulus or stiffness modulus approaches at low temperatures and high frequencies or short loading time, which is normally very close to 1 GPa in shear loading for most asphalt cements.

- **The steady-state viscosity, η_0**-- is indicative of the steady-state of asphalt cement which is asphalt specific.

- **The crossover frequency, ω_c, or crossover time, t_c**--the frequency at a given temperature where tan $\delta = 1$. At this point, the storage and loss moduli are equal. The crossover frequency can be considered as a hardness parameter that indicates the general consistency of asphalt at the selected temperature and is asphalt specific. The crossover frequency is the reciprocal of the crossover time, $t_c = 1/\omega_c$.

- **The rheological index, R**--the difference between the glassy modulus, G_g, and the dynamic complex modulus at the crossover frequency, $G^*(\omega_c)$. The rheological index is directly proportional to the width of the relaxation spectrum and indicates the rheological type. R is asphalt specific.

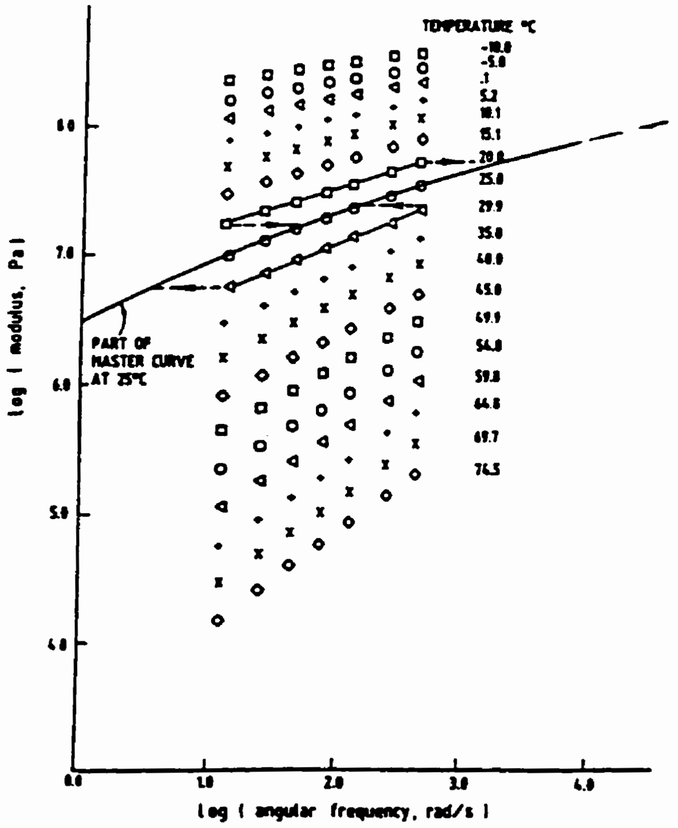


Figure 2.4 Time-Temperature Superposition Method for Building Master Curve (from Maccarroni S. 1987)

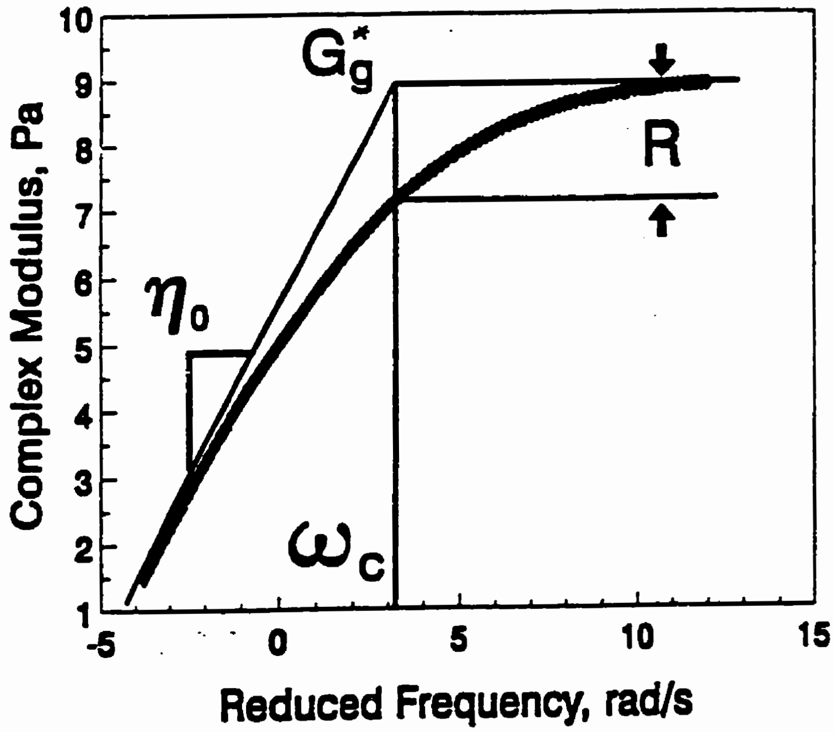


Figure 2.5 Master Curve Parameters (from SHRP A-002A study)

2.5.3 Mathematical Models for Asphalt Cements

To have a complete behavior of asphalt cement, as a linear visco-elasic material, it should be characterized at a wide range of loading times (frequencies) and temperatures. By expressing this behavior with a mathematical model, the responses can be reduced to a limited number of variables. Furthermore the mathematical model allows calculating the response of binders at loading times and temperatures other than those directly measured. The mathematical model of an asphalt binder could also be used for the prediction of pavement performance. A short review of the most important developed mathematical models for characterization of asphalt cement is presented below.

2.5.3.1 Jongepier and Kuilman Model

Jongepier and Kuilman (1969) suggested that the *relaxation spectra* for asphalt binders are approximately log normal in shape, and they derived a model for various rheological functions based on this assumption. In this model, the reduced frequency is replaced by the dimensionless frequency:

$$\omega_r = \omega\eta_0 / G_g \qquad [2.29]$$

where:

ω_r = reduced frequency, rad/s.

η_0 = the steady-state (Newtonian) viscosity, Pa.1

G_g = the glassy modulus, Pa.

The G′ (storage modulus) and G″ (loss modulus) can be calculated from the following formula:

$$G' = G^* \cos\delta = \int_{-\infty}^{\infty} H(\tau)\frac{(\omega\tau)^2}{1+(\omega\tau)^2}\, d\ln\tau \qquad [2.30]$$

$$G'' = G^* \sin\delta = \int_{-\infty}^{\infty} H(\tau) \frac{\omega\tau}{1+(\omega\tau)^2} d \ln \tau \tag{2.31}$$

where H(t) is the *logarithmic relaxation spectrum* and is expressed as a log normal distribution:

$$H(\tau) = \frac{G_g}{\beta\sqrt{\pi}} \exp - \left\{\frac{\ln \tau / \tau_m}{\beta}\right\}^2 \tag{2.32}$$

where the parameter τ_m, the exponential of the mean of the natural logarithmic of the relaxation times, is given by:

$$\tau_m = \frac{\eta}{G_g} \exp - \frac{\beta^2}{4} \tag{2.33}$$

where η is the zero-shear viscosity and β is the scale parameter for the log normal distribution.

Jongepier and Kuilman (1969) numerically integrated the above equations for a range of β values, producing a set of master curves for complex modulus and phase angles. These master curves were then compared to experimental data, and values for the various parameters were selected for a best fit. Jongepier and Kuilman reported that values predicted from this model fit the observed data to within the experimental error, but that the accuracy of the model was better for asphalts with similar β-values.

In the characterization of temperature dependence, or shift factors, Jongepier and Kuilman used the WLF (Williams, Landel, and Ferry, 1955) Equation:

$$\log a(T) = \frac{-C_1(T - T_R)}{C_2 + (T - T_R)} \quad [2.34]$$

where:

$a(T)$ = the shift factor relative to the defining temperature T

= $\eta_0(T)/\eta_0(T_R)$, $\eta_0(T)$ being the Newtonian viscosity at temperature T and $\eta_0(T_R)$ being the Newtonian viscosity at temperature T_R

C_1, C_2 = empirically determined constants

T_R = the defining temperature, which is a characteristic parameter for each binder.

The Jongepier and Kuilman model appears to be reasonably accurate and rigorous in its treatment of the linear visco-elastic properties of asphalt binders. Unfortunately, the use of this model requires the application of integral equations and/or transformations, which can only be solved using numerical methods. Additionally, the SHRP binder study indicates that the *relaxation spectra* of binders, although close to a log normal distribution at long relaxation times, significantly deviate from a log normal distribution for shorter loading times. An additional shortcoming of the Jongepier and Kuilman model, from a practical standpoint, is the mathematical complexity of its formulation, which makes it very difficult for routine practical applications in paving technology (SHRP A-369).

2.5.3.2 Dobson Model

Dobson (1969, and 1972) developed a mathematical model for describing the master curve, based on the empirical relationship between the phase angle and the modulus for asphalt binders. The fundamental assumption of Dobson's model is that the log-log slope of the complex modulus with respect to the loading frequency is a function of loss tangent and the width of the *relaxation spectrum*:

$$\frac{dy}{dx} = \frac{t}{(1+t)(1-0.01t)} \qquad [2.35]$$

where:

$y = \log(|G^*(\omega)| \, / G_g)$

$G^*(\omega)$ = the complex modulus at frequency ω

G_g = the glassy modulus

$x = \log(\eta_o \, \omega \, a(T) / G_g)$

η_0 = the steady-state or Newtonian viscosity

$a(T)$ = the shift factor at temperature T relative to the reference temperature

$t = \tan \delta$ (the loss tangent).

Additionally, Dobson (1972) observed a linear relationship between the loss tangent and the complex modulus, as expressed in the following equation:

$$\log(1+t) = -b \cdot y \qquad [2.36]$$

where b is a parameter proportional to the width of the relaxation spectrum, and y is as defined above.

Equations [2.35] and [2.36] can be mathematically combined to give Dobson's equation for relating the reduced frequency and complex modulus:

$$\log \omega_r = \log G_r - \frac{1}{b}\left[\log(1 - G_r^b) + \frac{20.5 - G_r^{-b}}{230.3}\right] \qquad [2.37]$$

where:

$\omega_r = \eta_o \omega a(T) / G_g$

$G_r = |G^*(\omega)| \, / G_g$

Like Jongepier and Kuilman, Dobson used the WLF Equation [2.34] to characterize the temperature dependency of asphalt binders. Dobson found that a single set of coefficients could be used to fit the shift factor data for a range of asphalt binders, but that different coefficients were needed for low and high temperatures.

It is difficult to assess the accuracy of Dobson's model since little comparison of measured and predicted moduli and phase angle was made in his paper. Additionally, the failure to express the modulus as an explicit function of reduced frequency is a serious drawback, as is the lack of a well-defined procedure for determining the constants in his equation for the modulus. However, Dobson's method for characterizing the temperature dependency appears to be reasonably accurate and, additionally, he presented a practical means for applying this method to rheological data on asphalt binders (SHRP A-369).

2.5.3.3 Dickinson and Witt Model

Dickinson and Witt (1974) developed a model in which the master curve of complex modulus is mathematically treated as a hyperbola. Several parameters can be calculated from the master curve using statistical methods, which characterize the master curve. The equation proposed for describing the variation in the complex modulus in terms of frequency is given below:

$$\log \left|G_r^*(\omega)\right| = 0.5\left\{\log \omega_r - \left[(\log \omega_r)^2 + (2\beta)^2\right]^{0.5}\right\} \qquad [2.38]$$

where:

$\left|G_r^*(\omega)\right|$ = the relative complex modulus at frequency $\omega = \left|G^*(\omega)\right| / G_g$

$\omega_r = \omega\, \eta_0\, a(T) / G_g$

η_0 = the Newtonian viscosity

$a(T)$ = the shift factor at temperature T relative to the selected reference temperature

β = *shear susceptibility* parameter, which is defined as the distance on a log-log scale between the glassy modulus and the modulus at $\omega_r = 1$.

Dickinson and Witt proposed the following similar equation for calculating the phase angle:

$$\delta(\omega) = \delta' + 0.25\,(\pi - 2\delta')\left\{\log\omega_r - \left[\left(\log\omega\right)^2 + \left(2\beta\right)^2\right]^{-0.5}\right\} \qquad [2.39]$$

where:

$\delta(\omega)$ = the phase angle at frequency ω, and

δ' = the limiting phase angle at infinite frequency.

The procedure for determining the coefficients of these equations involved an initial estimate of β and δ' by performing a linear regression on a linear version of the equation for the complex plane, which is:

$$\log\left|G^*{}_r\right| = -\beta\left[\frac{2(\delta - \delta')}{(\pi - 2\delta)}\right]^{1/2} \qquad [2.40]$$

The Dickinson model appears to be somewhat simpler than both the Jongepier and Kuilman or the Dobson model, and of similar or better accuracy. Dickinson and Witt reported log G_g ranged from 8 to 9.6 Pa, which is considerably different than the constant value of 9 Pa reported by other researches such as Van der Poel, 1954 and Pink et al., 1980. This fact suggests that there is some inaccuracy in both the glassy modulus and β parameters determined from this model.(SHRP A-369).

2.5.3.4 SHRP A-002A Model (Christensen and Anderson Model)

Anderson et al. (1994), under SHRP A-002A study, reviewed the current mathematical models and they developed functions for viscoelastic linear behavior of asphalt cements. The SHRP models used rheological properties that were based on the complex shear modulus and the phase angle of the asphalt cement. For the complex modulus, the following mathematical function was developed:

$$G^*(\omega) = G_g \left[1 + (\omega_c / \omega)^{(\log 2)/R}\right]^{-R/\log 2} \qquad [2.41]$$

and for the phase angle, the following related equation applied:

$$\delta(\omega) = 90 / \left[1 + (\omega / \omega_c)^{(\log 2)/R}\right] \qquad [2.42]$$

where:

$G^*(\omega)$ = complex dynamic modulus, in Pa, at frequency ω, rad/s.

G_g = glass modulus, typically 10^9 Pa.

ω_c = the crossover frequency, rad/s.

R = the rheological index

$\delta(\omega)$ = the phase angle, in degree, at frequency ω, rad/s.

By the combination of these two equations, the *rheological index* (R) is given by the following equation:

$$R = (\log 2) \log \left[G^*(\omega) / G_g\right] / \log(1 - \delta / 90) \qquad [2.43]$$

The linear visco-elastic (LVE) model developed by the SHRP A-002A research team is fairly accurate and complete, and is mathematically simple. By using a LVE model having only a few parameters, in SHRP mathematical asphalt cement model, it is possible to characterize rapidly an asphalt cement using limited testing with the degree of accuracy necessary for engineering design purposes. Additionally, the LVE model parameters have specific physical meanings, which allow them to be used in the development of rational physical-chemical property relationships. The model parameters can be used to estimate the rheological response under various critical conditions associated with the major mechanisms of pavement distress (Christensen 1992).

Christensen (1992) used the WLF equation [2.34] for temperatures above T_R and and an Arhennius function, Eq.[2.44], for temperatures below the defining temperature for describing the temperature dependency of asphalt cement:

$$log\, a(T)_R = \frac{2303 E_a}{H\left(\frac{1}{T} - \frac{1}{T_d}\right)} \qquad [2.44]$$

where:

E_a is the activiation energy for flow below T_R

H *is* the ideal gas constant, 8.34 J/mol-°K.

Christensen (1992) suggested that the values for constant parameters in the WLF equation, C_1 and C_2, equal to 19 and 92 respectively.

2.5.3.5 Other Studies

De Bats et al. (1983) evaluated various models for predicting the dynamic mechanical properties of asphalt cements. They found that the Jongepier and Kuilman model fit much better than the Dickinson and Witt model. Maccaroni (1987) studied the dynamic mechanical properties of a large number of asphalt cements. Macarroni determined that the Dickinson and Witt model fit the dynamic response of asphalt cements quite well. Maccaroni reported typical standard errors of fit for the Dickinson and Witt model ranged from about 0.004 to 0.008 on a log scale.

2.6 LOW-TEMPERATURE CHARACTERIZATION OF ASPHALT

It is generally known that the behavior of asphalt paving mixtures under low-temperature conditions of induced stress is affected by the response of the asphalt cement. During the service life of the pavement, asphalt is exposed to low temperatures, which tend to alter its rheological response. Many researchers (Special Report 91-5, U. S. Army Corps of Engineers, 1991) have agreed that asphalt cement is the dominant component for low-temperature performance of the asphalt pavement mixtures. Therefore, it is very important to study the low-temperature behavior of asphalts to obtain a better understanding of factors that affect the low-temperature behavior of pavement.

Some researchers have taken different approaches for the characterization of the asphalt cement at low-temperatures. A short review of characterization of the asphalt cement at low temperatures is presented below (Kandhal et al. 1988):

Penetration measurements at low temperatures were used by Shoor et al. (1966) to determine the glass transition temperature of paving asphalts. Heukelom (1966) related the asphalt stiffness to its penetration value. He also introduced the modified asphalt Penetration Index (PI), which is a measurement of the temperature susceptibility and is used to determine the asphalt stiffness at low temperatures. Hicks et al. (1993), in a SHRP binder validation study, found that the penetration of asphalt cement at 15°C is a good indicator of the fracture temperature of the mixture.

Doyle (1958) measured the ductility of various asphalts at 12.8°C, regardless of sources, and observed extensive pavement cracking when the ductility dropped below 5 cm. According to Traxler (1961), low ductility values are demonstrated by asphalt with a greater degree of complex flow. Welborn et al (1966), and Kandhal and Wenger (1975) reported a good correlation between the ductility and the shear susceptibility at 7° and 15.6°C. A study by Kandhal and Koehler (1984) of six experimental test sections in

Pennsylvania indicated that lower ductility were associated with a higher incidence of load-associated longitudinal cracking.

Schweyer et al. (1977) have done considerable work on the use of the capillary rheometer and the development of several generations of the constant-stress rheometer which have been used to examine both asphalt stiffness and viscosity at low temperatures. Of the methods that are available for determining the low-temperature viscosity, the most widely used are probably the cone-plate viscometer or the sliding-plate viscometer.

Puzinauskas (1967) found that the low-temperature viscosities of asphalt cements vary extremely over a wide range. This range increases with decreasing temperatures. Generally, poor correlation was registered between the consistency properties such as viscosity, penetration, or ductility. Furthermore, measurements at high or moderate temperatures cannot be used to predict the behavior of asphalts at subfreezing temperatures.

Some researchers have used the glass transition temperature (T_G) for low-temperature characterization of asphalt cements. T_G represents a sudden change in a physical property as the material passes from a solid state to a fluid state. Glass transition temperature is usually determined by one of three methods: penetration versus temperature, volume versus temperature, or differential thermal analysis. Dilatometric methods were applied to determine T_G of asphalts by Schmit and Barrall (1965), Schmidt et al. (1965), Schmidt and Santucci (1966), and Jongepier and Kuilman (1969). Schmidt et al.(1965) investigated 52 Bureau of Public Roads asphalts and found T_G range from -17.2° to -32.8°C. Jongepier and Kuilman (1969), using dilatometry, concluded that the "universal" constants in the WLF equation were seen to vary with stress frequency.

Majidzadeh and Schweyer (1967) evaluated the viscoelastic response to stress near T_G. At -8.9°C, the four asphalts examined exhibited an instantaneous elastic deformation. No such deformation was observed at higher temperatures between -3.9° and 5°C. Dobson (1969) argued that difficulties in applying the WLF equation to asphalt are due to the small values of T_G. Schweyer (1974) examined the effect of pressure on T_G and concluded that there is a linear relation between T_G and pressure.

Haas and Anderson (1969) demonstrated that asphalt source or type could have a major influence on low-temperature cracking of an asphalt layer. They used a thin film stress-strain technique as a criterion for cracking when the stiffness at 1000 seconds reaches 1.4×10^8 N/m^2 at a cooling rate of 5°C/hr.

Pagen (1965, 1967) conducted dynamic and creep tests over a range of temperatures from -18° to 52°C. His experimental data indicated that both the time-temperature superposition principle and the linear viscoelastic theory are applicable to the asphalt concrete tested at a satisfactory level of approximation.

Majidzadeh and Schweyer (1967) studied the viscoelastic response of four asphalts in the temperature range of -9° to 5°C using cylindrical specimens. At low temperatures such as -9°C, the asphalts exhibited some instantaneous elastic deformation, which is represented by the spring in the Maxwell element. Subsequently, they studied (1968) the viscoelastic response of aged asphalt cements. Jongepier and Kuilman (1969) determined the linear viscoelastic properties of a number of asphalt cements, ranging from extreme sol to extreme gel types, at various frequencies and over a temperature range of -20° to 160°C. Pink et al. (1980) used a Rheometric Mechanical Spectrometer (RMS) to make accurate low-temperature viscoelastic measurements on asphalts down to -94°C. A dynamic master curve was developed to separate the effect of time and temperature. Button et al. (1983) also used the RMS to measure viscosity of asphalts from 0° to -46°C.

In asphalt rheology, the use of stiffness or resistance to stress as a function of time dates from Nijbore and Van der Poel's (1953) description, although Nutting (1921, 1921, 1943) proposed a generalized stress-strain-time concept in 1921. The Shell sliding-plate rheometer developed by Fenijn and Kroosh (1970), conveniently measures low-temperature asphalt stiffness. Values referred to as *limiting stiffness* and defined as the asphalt stiffness modulus above which pavement cracking is imminent have been reported by Van der Poel (1954), Fromm and Phang (1970), Readshaw (1972), and Gaw (1978). These values ranged from 1.38×10^8 to 5×10^8 N/m^2 at a loading time of 10000 seconds. Other researchers (Hills and Brien 1966; Burgess et al. 1971; and Haas 1973) have established methods to predict pavement cracking based on *fracture temperature*, which is a temperature at which a restrained beam of asphalt concrete cracks when cooled at a certain rate.

Bahia (1992) studied the low-temperature rheological and volumetric properties of eight different paving grade asphalts. He introduced the concept of "physical hardening" in terms of the change in creep response as a function of isothermal time for studied asphalt cements. Physical hardening results in significant changes that must be considered in any attempt to characterize the low-temperature behavior of asphalt cement. Physical hardening is sensitive to isothermal temperature, and proceeds at a rate that decreases rapidly with isothermal age but it is not affected by oxidation aging and presence of mineral fillers.

2.7 ASPHALT GRADING SYSTEMS

There are three commonly used systems, based on the consistency tests, for the grading of asphalt cements. These traditional grading systems are penetration, viscosity, and Aging Residue (AR) system. A short historical review of these systems is given in the next pages.

Bowen (1888) invented the Bowen penetration machine to determine the consistency of asphalt cement. In early 1900, the Bureau of Public Roads (now the Federal Highway Administration), and the American Society for Testing and Materials

(ASTM D 946) were instrumental in making the penetration test a standard for controlling consistency of paving asphalt cements. After several modifications of the initial penetration machine, by 1910 the penetrometer became the principal means of measuring and controlling the consistency of semi-solid asphalts.

The American Association of State Highway Officials (AASHO) published the standard for penetration graded asphalt cement in 1913. In 1918, the Bureau of Public Roads introduced the penetration grading system by developing various penetration grades suited to different climatic conditions and applications. In the penetration grading system, asphalt cement is specified by the minimum and maximum penetration tests at 25°C such as 120-150 or 200-300.

The next major change in asphalt cement grading specification was initiated in the early 1960s by FHWA, ASTM, AASHTO, industry and state highway departments that wanted the asphalt cements to be graded by viscosity at 60°C. The primary objectives of the viscosity grading system were to replace the empirical penetration test with a rational scientific viscosity test and to measure the consistency at 60°C (rather than 25°C). This temperature approximates the maximum pavement temperature on a hot summer day in most parts of the United States. ASTM D 3381 and AASHTO M 226 standardized asphalt cements based on viscosity on 60°C. Various grades were developed to suit the different climatic conditions and applications, for example AC-2.5 for very cold conditions and AC-40 for very hot climate conditions.

During the early 1960s when the viscosity grading system was being developed, the California Department of Highways developed a parallel Aged Residue (AR) viscosity grading system with the cooperation of the Pacific Coast User Producer Group. AR viscosity grading was based on the viscosity of the aged residue resulting from the rolling thin film oven (RTFO) test rather than the viscosity of the original asphalt cement as recommended by the FHWA. For example, AR-1000 represents a viscosity of 1000 poises at 60°C of the RTFO aged residue (Roberts et al. 1991).

2.8 CRITICAL REVIEW OF ASPHALT CEMENT SPECIFICATIONS

A primary drawback of the penetration/viscosity specifications is that they extrapolate the high and intermediate pavement testing temperatures for low-temperature characterization of asphalt cements. Many researchers have shown that this extrapolation is not accurate. This limitation is especially important for the proper selection of asphalt binders in cold climate regions. Additional shortcomings of the current penetration-viscosity specifications include:

- specifications are limited to only virgin asphalt binders
- long term aging of the asphalt cement over time not considered
- fundamental asphalt cement properties that directly relate asphalt pavement mixture properties to long term pavement performance are not addressed.

Because the rheological properties of the asphalt cement are a function of both loading time and temperatures, *temperature susceptibility* parameters must be based on measurements at different temperature but with similar loading times. Otherwise, the *temperature susceptibility* parameter will conflict with loading time as is the case when penetration and viscosity or softening point and penetration measurements are combined to calculate PI or PVN. Anderson et al. (1992) concluded that the empirical temperature susceptibility indices, (PI and PVN), used in conjunction with the various versions of Van der Poel nomograph, did not provide accurate estimates of the limiting stiffness temperature.

The inherent limitations of the current characterization and grading systems based on the viscosity and penetration methods have identified the need for new Strategic Highway Research Program (SHRP) asphalt pavement performance test procedures and asphalt binder specifications.

2.9 STRATEGIC HIGHWAY RESEARCH PROGRAM (SHRP)

The Strategic Highway Research Program (SHRP) was established by the United States Congress in 1987 as a five-year, 150 million dollar research program to improve the overall performance and durability of roads and highways. SHRP research efforts were focused on four primary areas:

- Highway operations
- Concrete and structures
- Asphalt binder and asphalt mixture properties
- Long-term pavement performance.

2.9.1 Performance Graded (PG) Binder System

One of the primary objectives of the SHRP was to develop performance-based specifications for asphalt mixtures, i. e., specification that would relate directly to pavement performance. SUPERPAVE™ is a performance-based asphalt mix design procedure, developed by SHRP, to replace the current empirical asphalt mixture design procedures (i.e., Marshall and Hveem). SUPERPAVE™ includes a new specification for asphalt binders called Performance Graded (PG) binder system, a performance-based asphalt mixture design, a materials database, and a weather database for more than 1600 stations throughout North America.

The SHRP asphalt binder test procedures include:

- Dynamic Shear Rheometer (DSR): tests asphalt cement susceptibility related to rutting and fatigue
- Bending Beam Rheometer (BBR): tests asphalt cement susceptibility related to low temperature cracking
- Direct Tension Test (DTT): tests asphalt cement susceptibility related to low temperature cracking.

The Dynamic Shear Rheometer (DSR) is used to measure the complex shear modulus (G^*) and the phase angle (δ) of the asphalt binder under repeated sinusoidal load. G^* is a measure of the asphalt cement resistance to deformation. Phase angle is the time delay between the applied stress/strain and the response strain/stress. Figure 2.6 depict the stress and strain, G^* and δ from DSR test. In the PG asphalt binder specifications, $G^*/\sin\delta$ indicates the degree of susceptibility to permanent deformation and $G^*\cdot\sin\delta$ indicates the degree of susceptibility to fatigue (AASHTO TP 5).

The Bending Beam Rheometer (BBR) evaluates the low temperature creep stiffness properties of the asphalt cements. The outputs of the BBR test are the creep stiffness (S), and the m-value. Figure 2.7 illustrates the m-value, which is the slope of the log of the stiffness versus loading times at 60 seconds loading times. The BBR gives an indication of the asphalt pavement's susceptibility to low temperature cracking (AASHTO TP 1).

The Bending Beam Rheometer (BBR) does not provide information regarding stress/strain of the asphalt at failure. Therefore, SHRP researchers developed the Direct Tension Test (DTT) to determine low temperature stiffness properties of the asphalt cement at failure (i.e., tensile strain at failure ε_f). DTT is compulsory for the binders that do not satisfy the m-value criterion as determined by the Bending Beam Rheometer (AASHTO TP 3).

Durability of the asphalt cement is determined by accelerated aging in the lab. Aging of asphalt cement results from two primary mechanisms: volatilization and oxidation. Volatilization is the loss of light constituents during heating in the mix plant, transport and laydown. Oxidization is the reaction of the binder with oxygen in the surrounding environment during the long-term service of pavement. There are two procedures outlined by the PG binder system to simulate the aging of asphalt binder: the

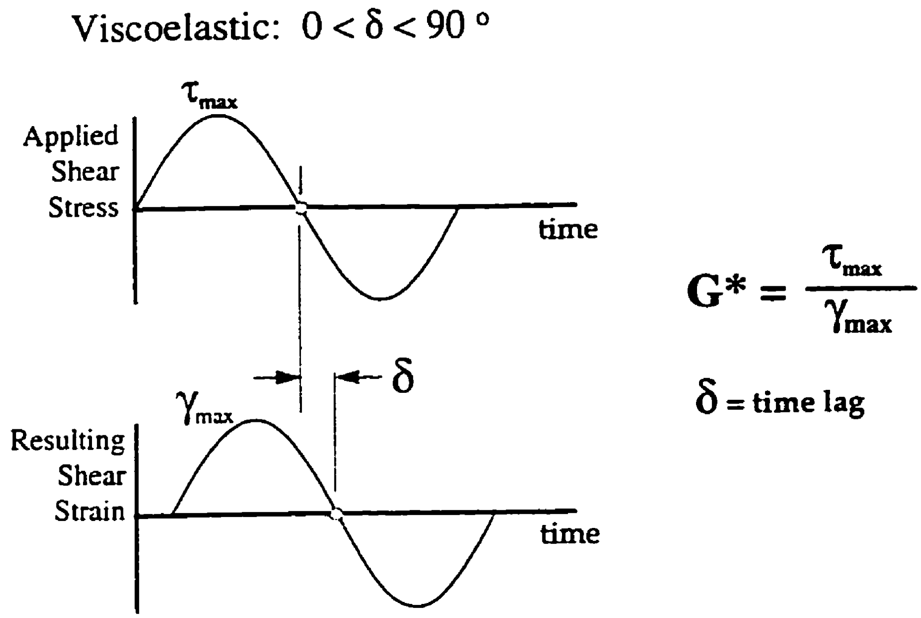


Figure 2.6 Stress-Strain Responses of Asphalt Cement with DSR Test (from National Asphalt Training Center, Project 101)

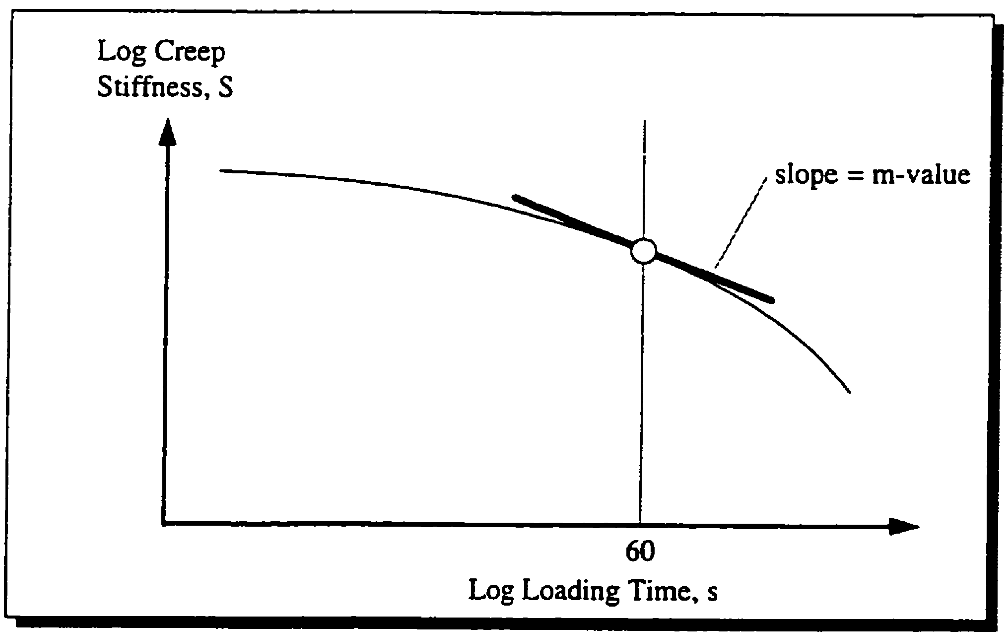


Figure 2.7 The m-Value from BBR Test

Rolling Thin Film Oven (RTFO) for short-term volatilization simulation, and the Pressure Aging Vessel (PAV) to simulate the long-term oxidization aging.

RTFO, which is detailed in AASHTO T 240 and ASTM D 2872 exposes a thin film of the asphalt binder to heat and air to simulate the asphalt binder aging during the production and construction of the asphalt pavement. The RTFO aging method was developed by the California Highway Department.

The PAV simulates the long term aging of the asphalt binder that results from oxidization in the pavement during the pavement's service life after construction. The PAV exposes the asphalt binder samples, that have been short term aged in the RTFO to heat and pressure to simulate, in a matter of hours, years of in-service aging. Therefore, the asphalt residue from RTFO and PAV represents a binder that has been exposed to all conditions to which the binder is subjected during production and in-service.

2.9.2 PG Binder Criteria for Pavement Distresses

The PG binder system has selected four parameters related to the main pavement distresses such as fatigue, rutting and low-temperature cracks. These parameters are $G^*/\sin\delta$, $G^*.\sin\delta$ corresponding to the rutting and fatigue and S and m-value corresponding to the low-temperature cracks.

There are different opinions about the contribution of the binder to the rutting resistance. It is a fact that soft asphalts are not used to construct pavements in hot desert climates. It is also a fact that, during the past decade, more and more engineers are specifying polymer-modified binders, at a much higher cost, to mitigate rutting problems. Aggregate properties are without doubt very important. However, many researchers agree that it is not good engineering practice to ignore the binder properties. To minimize the rutting, the work dissipated during each loading cycle should be minimized. For a viscoelastic material, the work dissipated per cycle (W_c) is calculated in terms of stress (σ) and strain (ε) as follow:

$$W_c = \pi \,.\, \sigma \,.\, \varepsilon \,.\, \sin\delta \qquad [2.45]$$

Rutting within the asphalt concrete layer can be assumed as a stress controlled (σ) repetitive phenomena. Therefore, the following substitution can be made:

$$W_c = \pi \,.\, \sigma_o \,.\, \varepsilon \,.\, \sin\delta \qquad [2.46]$$

since $\varepsilon = \dfrac{\sigma_o}{G^*}$

$$W_c = \pi \,.\, \sigma_o^2 \,.\, \left[\frac{1}{G^* / \sin\delta}\right] \qquad [2.47]$$

The above relationship indicates that the work dissipated per loading cycle is inversely proportional to the parameter of the $G^*/\sin\delta$, which is the parameter which has been selected to control rutting in the PG specification.

Fatigue of a pavement can be a controlled-stress distress phenomenon (typical for thick pavement layers) or a controlled-strain phenomenon (typical for thin pavement layers). Fatigue cracking, however, is known to be more prominent in the pavements with thin layers. Based on this assumption, fatigue can be considered as predominantly a strain controlled phenomenon. Therefore, the dissipated work concept can be used to derive the parameter used in the PG specification. For a strain-controlled cyclic loading, the work per cycle equation can be rewritten as follows:

$$W_c = \pi \,\sigma\, \varepsilon .\sin\delta \qquad [2.48]$$

where ε_o is the strain amplitude being applied. Since the stress (σ) is related to the strain by G*:

$$\sigma = \varepsilon_0 . G^* \qquad [2.49]$$

Substituting leads to the following equation which shows that W_c, under the strain controlled conditions, is directly related to the G* sin δ:

$$W_c = \pi \, \varepsilon_0^2 . [G^* . sin\delta] \qquad [2.50]$$

To prevent fatigue, it is therefore best to limit the energy dissipation by limiting the value of the parameter of G*. sin δ (Bahia and Anderson 1995).

The PG performance criteria need more validation and some researchers have shown that the PG fatigue criterion has a very low correlation with mixture and field fatigue performance. This can be attributed to higher influences of other mixture and pavement parameters such as aggregate, asphalt content, and pavement layers thickness and lower influences of asphalt cements in fatigue.

2.9.3 The PG Binder Specifications

The Performance Graded (PG) asphalt cement specifications are based on performance criteria at various temperatures corresponding to local climate. In the PG binder system, each binder is graded based on high and low pavement design temperature. For example, the PG 58-34 is a binder that has satisfied the PG criteria at 58° C for maximum and -34° C for minimum pavement temperatures.

The procedure for selecting a Performance Graded (PG) asphalt binder for a particular climatic region is as follows:

- Determine the maximum ambient air temperature for the area with a corresponding reliability.
- Determine the maximum pavement temperature corresponding to the ambient air temperature based on an algorithm from the SUPERPAVE™ software.
- Determine the minimum ambient air temperature with a corresponding reliability.
- The minimum pavement temperature is equal to the minimum ambient air temperature.

The PG asphalt cement specification has been approved by the AASHTO as a provisional standard (AASHTO MP-1) and is shown, in part, in Table 2.2.

2.9.4 Critical Review of PG Binder System

The PG system assumes the asphalt binder to be a linear visco-elastic material that displays different visco-elastic properties at varying temperatures. For a complete characterization of the stress-strain-time-temperature response of typical asphalt cement, with a time-temperature superposition concept, a master curve and its associated temperature shift factors were used by the SHRP binder studies. The time dependency of the asphalt cement is reflected in the master curve whereas the temperature dependency is reflected in the temperature shift factor (Anderson et. al., 1991).

There is an agreement between researchers, highway agencies, and suppliers that the PG binder system is an improvement for characterization and grading of asphalt cements comparing to the traditional testing and grading methods. The PG binder system is still under development; therefore, there are some challenges in the implementation of this new grading system, which should be understood. Some of the challenges for switching from viscosity, penetration system to the PG binder system are:

- Repeatability and reproducibility of the PG binder tests
- Difficulties in the direct tension test

Table 2.2 AASHTO Performance Graded Binder Specification (MP1)

Performance Grade	PG 52							PG 58					PG 64				
	-10	-16	-22	-28	-34	-40	-46	-16	-22	-28	-34	-40	-16	-22	-28	-34	-40
Average 7-day Maximum Pavement Design Temperature, °C	<52							<58					<64				
Minimum Pavement Design Temperature, °C	>-10	>-16	>-22	>-28	>-34	>-40	>-46	>-16	>-22	>-28	>-34	>-40	>-16	>-22	>-28	>-34	>-40
	Original Binder																
Flash Point Temp. T48: Minimum °C	230																
Viscosity, ASTM D 4402: Maximum, 3 Pa·s (3000 cP), Test Temp, °C	135																
Dynamic Shear, TP5: G*/sin δ, Minimum, 1.00 kPa Test Temperature @ 10 rad/s, °C	52							58					64				
	Rolling Thin Film Oven (T240) or Thin Film Oven (T179) Residue																
Mass Loss, Maximum, %	1.00																
Dynamic Shear, TP5: G*/sin δ, Minimum, 2.20 kPa Test Temp @ 10 rad/sec, °C	52							58					64				
	Pressure Aging Vessel Residue (PP1)																
PAV Aging Temperature, °C	90							100					100				
Dynamic Shear, TP5: G*sin δ, Maximum, 5000 kPa Test Temp @ 10 rad/sec, °C	25	22	19	16	13	10	7	25	22	19	16	13	28	25	22	19	16
Physical Hardening [c]	Report																
Creep Stiffness, TP1: S, Maximum, 300 MPa m-value, Minimum, 0.300 Test Temp, @ 60 sec, °C	0	-6	-12	-18	-24	-30	-36	-6	-12	-18	-24	-30	-6	-12	-18	-24	-30
Direct Tension, TP3: Failure Strain, Minimum, 1.0% Test Temp @ 1.0 mm/min, °C	0	-6	-12	-18	-24	-30	-36	-6	-12	-18	-24	-30	-6	-12	-18	-24	-30

- Suppliers' concerns
- Low temperature algorithm for binder selection
- Training
- Time and cost of testing
- Quality Control/Quality Assurance for binders with the PG system
- Extension of the implementation for modified asphalt and asphalt recycling.

More details related to these challenges have been reported recently for Saskatchewan Highways and Transportation (Soleymani and Gerberandt 1997). This dissertation looks to the implementation of the PG binder system for asphalt pavement recycling and intends to characterize and model the blended binder with the PG system.

2.10 RECYCLING AGENTS FOR ASPHALT PAVEMENT RECYCLING

Pavement recycling, reuse of the reclaimed asphalt pavement with or without adding new asphalt cement and aggregate, has been developed widely for rehabilitation of pavements in the last two decades. The first report of using old materials or asphalt pavement recycling dates back to 1915 but this technique was not used widely until 1970 (Taylor 1978). The oil embargo in 1973 and the subsequent increases in crude oil prices were the main reasons for serious application of asphalt pavement recycling. Decrease of highway maintenance budgets, restrictions on the use of new aggregates, and the lack of virgin aggregate sources in some areas, and environmental considerations have caused asphalt pavement recycling to become a popular method for the rehabilitation of pavements.

The primary advantages of asphalt pavement recycling may be summarized as follows:

- Conservation of aggregate
- Conservation of asphalt binder
- Conservation of energy
- Minimal environmental impact resulting from the production of new

pavement materials

- Elimination of the clearance problems associated with overlaying the existing pavement in restrictive highway geometry such as vertical clearance at bridges, signs, and tunnels, and realignment of curb heights and manholes, etc.

Recycling agents are added to the reclaimed asphalt pavement to:

- Restore the recycled asphalt to a suitable consistency for construction
- Restore optimal chemical and physical characteristics to the aged asphalt
- Provide sufficient binder content to satisfy the mixture design requirements.

Recycling agent does not instantaneously combine with the old asphalt cement that coats the salvaged material. A diffusion process takes place over a period of time during which this combination occurs. Some researchers studied this phenomenon. Carpenter and Wolosick (1980) studied the influence of the diffusion process on the recycled material and showed that the diffusion process exerts a large influence on the material properties. Noureldin (1987) used a partial extraction technique that had the effect of dividing the blended asphalt film into microlayers. The binder recovered from each microlayer was characterized by means of consistency distribution of the binder film around the aggregate. He developed a model for the prediction of the viscosity or penetration of the whole blended asphalt film based on the extracted viscosity or penetration of the four microlayers of the blended asphalt.

2.10.1 Current Methods for Selection of Recycling Agent

Asphalt researchers and highway agencies have used different methods to select the type and proper amount of recycling agents for asphalt pavement recycling projects. Current recycling design procedures use viscosity and/or penetration measurements of the recycling agent to select the type and amount of recycling agent required to meet a specific grade for the recycled mixture. The most common guideline, ASTM D 4887, uses the viscosity-blending chart. ASTM D 2172 and ASTM D 1856 standard test methods have been used widely to separate asphalt cement from the aggregate and recover aged asphalt binder from the solution. The viscosity and penetration of the aged asphalt binder and recycling agent are determined. Blending charts are used to determine the percentage of the recycling agent and the viscosity or penetration of the blended binder. Figure 2.8 depicts the viscosity chart for the selection of the recycling agent.

Davidson and William (1977) used a mathematical formula based upon the aggregate gradation to calculate the probable asphalt demand in the recycled mixture. In this method, the minimum amount of necessary recycling agent equals the calculated asphalt demand minus the asphalt content of the pavement determined by extraction. They suggested that a recycling agent with a composition parameter (N+A1)/(P+A2), of 0.4 to 0.8 and N/P ratio grater than 0.5 will extend the life of most aged asphalts without exudation. N, P, A1, and A2 represent nitrogen bases, paraffins, first acidaffins and second acidaffins of the recycling agent. Davidson and his co-workers used viscosity and penetration charts for selecting the recycling agent and the design of recycled mixtures. A study by Dunning and Mendenhall (1977) suggested a viscosity range of 90 to 300 MPa/s (cp) at 60°C for the physical properties of recycling agent. Dunning and Mendenhall suggested that recycling agents contain polar and aromatic compounds and minimum paraffin or saturated content for the chemical compatibility. McLeod (1985, 1989) suggested that, because the Penetration Viscosity Number (PVN) remains constant, regardless of the time and temperature, it can be used for the selection of asphalt for a recycling project based on the original PVN of the asphalt. This method is unique because it does not depend on single point measurements, instead it uses temperature susceptibility parameters to control critical properties.

Davidson and Mc Innis (1989) applied the PVN concept to the selection of a recycling agent for recycling laboratory mixtures. The laboratory blending study indicated a potential problem when conventional soft asphalts are used with 50% RAP in

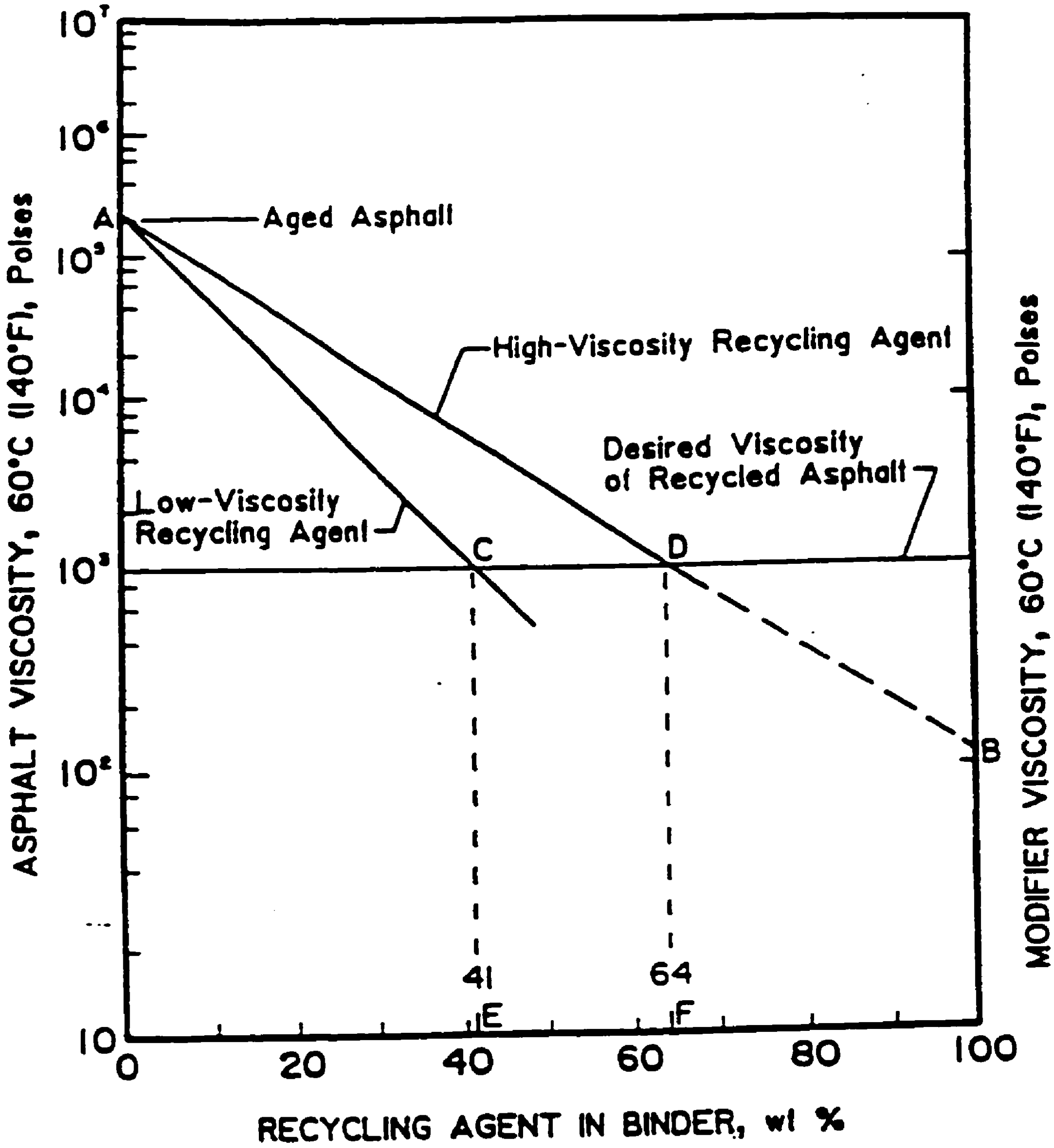


Figure 2.8 The Viscosity Chart for Determining the Type /Amount of Recycling Agent (ASTM D 4887)

regions with freezing indices greater than 2000. In this study, a recycled pavement using polymer-modified asphalts satisfied the McLeod criteria for that cold condition. Stamatinos (1988) suggested the concept of *apparent penetration* that allows the design of high ratio, recycled asphalt concrete mixes to suit the needs of the project in terms of final field penetration. Some agencies are using one or two grades softer asphalt binders than the original asphalt binder in the reclaimed mixture.

In conclusion, these methods have shown mixed results. The lack of one preferred method that has shown a high success means that the majority of recycling jobs are being designed based on trial and error procedures that will use experience and blending charts to select the mixture design.

2.10.2 Models for Blended Binders

A blended binder, composed of an aged binder and a recycling agent, can be considered as a binary liquid mixture. Many investigators studied the properties of blended binder based on viscosities of components since the early work of Arrhenius (1887):

$$ln\eta = v_1\, ln\eta_1 + v_2\, ln\eta_2 \qquad [2.51]$$

where v_1 and v_2 are the volume fractions, and η_1 and η_2 the viscosities of the two liquids.

Irving (1977) examined more than 50 other equations proposed by different researchers for characterization of binary liquid and concluded that the Grunberg and Nissan Equation (1949) provided best overall mixing rule in terms of accuracy and simplicity:

$$ln\eta = x_1\, ln\eta_1 + x_2\, ln\eta_2 + x_1 x_2 G_{12} \qquad [2.52]$$

The interaction G_{12} is often considered a constant, however G_{12} may be a function of x_i, where x_i may be mole, mass, or volume fraction. Mehrotra (1990) has used the Grunberg equation to model bitumen/gas and bitumen/solvent systems. However, very little effort has been focused on using this equation to predict the viscosity of blended binders. Chaffin et al. (1995) examined the Gurunberg equation for aged asphalt and recycling agent mixtures and concluded that the interaction parameters G_{12} is a strong function of the viscosity difference between the aged asphalt and recycling agent. They developed Dimensionless Log Viscosity (DLV) for viscosity normalization. All current models characterizing blended binders are based on the viscosity/penetration of the components. No literature was found relating to characterization of blended binders based on rheological parameters, such as complex modulus or phase angle. One of the main objectives of this dissertation is to develop some models for the characterization of blended binders based on rheological parameters.

2.11 CHAPTER SUMMARY

Asphalt cement could be characterized as a linear viscoelastic material. For a complete characterization of asphalt cement, the responses of asphalt cement should be measured in a full range of construction and pavement service temperatures and traffic loading frequencies.

The current grading system, based on consistency measurements, has some shortcomings for the characterization of asphalt cements especially at low pavement temperatures. The dynamic creep and the mechanical analysis, including Dynamic Shear Rheometer and Bending Beam Rheometer could characterize asphalt cements at different pavement temperatures and loading frequencies. The current methods for selection and characterization of virgin asphalt cement and blended binder are based on consistency measurements and cannot predict directly the performance of the recycled asphalt mixtures.

Of the various rheological models for characterizing the stress-strain behavior of the asphalt cement, the SHRP A-002 model seems more accurate and easier for engineering applications. The time-temperature superposition principal is valid when used for characterizing asphalt cements.

The PG binder system is an improvement for characterization and grading of asphalt cements but it is necessary to have more research in this area to address the challenges in the complete implementation of the PG binder system. This research is an effort in implementation of the PG binder system for design of asphalt recycled mixtures.

CHAPTER THREE

MATERIALS, TESTING EQUIPMENT, AND RESULTS

3.1 INTRODUCTION

The physical properties of the materials and the experimental design used in this research are described in this chapter. The laboratory testing equipment, including the Dynamic Shear Rheometer (DSR) and the Bending Beam Rheometer (BBR) are explained. The testing results of the DSR and BBR and some discussions regarding the potential sources of error for test results are presented.

The objectives of this study were to:

1. Characterize blended binders with cyclic (DSR) and creep (BBR) tests.
2. Develop models for prediction of blended binders properties based on DSR and BBR parameters (G^*, δ, S, and m-value).
3. Predict the performance of blended binders with PG binder criteria.
4. Review procedures for the selection of recycling agents for asphalt pavement recycling based on developed models.
5. Characterize blended binders with their master curves.

3.2 MATERIALS

To accomplish these objectives, one asphalt cement, a 150-200 penetration grade supplied by Moose Jaw Asphalt Inc., was selected as the *original binder* in the Reclaimed Asphalt Pavement (RAP). This typical asphalt cement is used for asphalt pavement in cold regions in Canada. Table 3.1 shows the physical properties of the 150-200-penetration grade asphalt cement used in this study.

Four different recycling agents, consisting of two soft asphalt cements and two oil-based materials, were selected for rejuvenating the aged original binder. The two soft asphalt cements were 200-300 and 300-400 penetration grades supplied by Husky Oil Ltd. from their Lioyminster Refinery. Table 3.2 shows the physical properties and Figure 3.1 depicts the relationship of viscosity versus temperature for the soft asphalt cements. The ductility test results, in Table 3.2, for two soft asphalts (200-300 and 300-400 penetration grade) were the same. This means that the ductility test could not differentiate between two soft asphalts. The highest temperature that the asphalt cements were characterized was 135°C, with viscosity test, and the lowest temperatures was 0°C, with penetration test. There was not any characterization for temperatures lower than 0°C. The PVN for 150-200, 200-300, and 300-400 asphalt cements were -0.17,-0.05, and 0.01 respectively. This means that the 150-200 and 300-400 asphalt cements were the highest and l;owest temperature susceptible asphalt cements respectively.

One of the two recycling agents used in this study was a petroleum hydrocarbon, supplied by Witco Corporation, California, U.S.A., with the product name "Cyclogen L," and will be referred to *Cyclogen* in this dissertation. Some technical information for this material is given in Table 3.3. The other oil based material with the product name "Shell-Flex 210", supplied by Shell Canada Products Limited and is called *Flexon* in this dissertation, was used as the other modifier in this study. Flexon includes both paraffinic and naphthenic oils. Table 3.4 shows some physical properties of Flexon.

Table 3.1 Physical Properties of the 150-200 Asphalt Cement (Original Binder)
Reported by Supplier (Moose Jaw Asphalt Inc.)

Properties	Test Method	Value
Absolute Viscosity @60°C(Pa.s)	ASTM D 2171	173
Kinematic Viscosity @135°C, mm^2/s	ASTM D 2170	380
Penetration, 25°C, 100g/5s, dmm	ASTM D 5	155
Penetration, 4°C, 200g/5s, dmm	ASTM D 5	64
Toughness, J	Benson	8
Tenacity, J	Benson	7
Softening Point, ° C	D 3686	47
RTFO Loss on Heating %	AASHTO 240	1.2
Tests After RTFO	ASTM D 2171	
Absolute Viscosity @60°C (Poise)	-	5205
RTFO Viscosity/Original Viscosity	ASTM D 5	3.0
Penetration @25°C, 100g/5s, dmm		74

Table 3.2 Physical Properties of 200-300 and 300-400 Asphalt Cements
Reported by Supplier (Husky Oil Operations Ltd.)

Properties	Test Method ASTM	200-300 Asphalt	300-400 Asphalt
Density @15°C, kg/L	D 70	1.0250	1.0210
Pen. @25°C, 100g / 5 s.	D 5	255	365
Pen. @0°C, 200g / 60 s.	D 5	76	112
Flash Point, °C	D 92	262	248
Ductility @25°C, cm	D 113	150+	150+
Solubility in C_2HCl_3, % mass	D 2042	99.9	99.9
Viscosity @60°C, Pa.s	D 2171	48	31
Viscosity @135°C, mm^2/s	D 2170	211	166
Softening point, °C	D 36	36	31
Thin film oven test, % loss	D 1754	0.59	0.70
Test after TFOT			
Pen. @ 25°C, 100g / 5 s.	D 5	130	181
Pen. @0°C, 200g / 60 s.	D 5	43	54
Absolute Viscosity @60°C, Pa.s	D 2171	131	78.3
Kinematic Viscosity @135°C, mm^2/s	D 2170	330	229

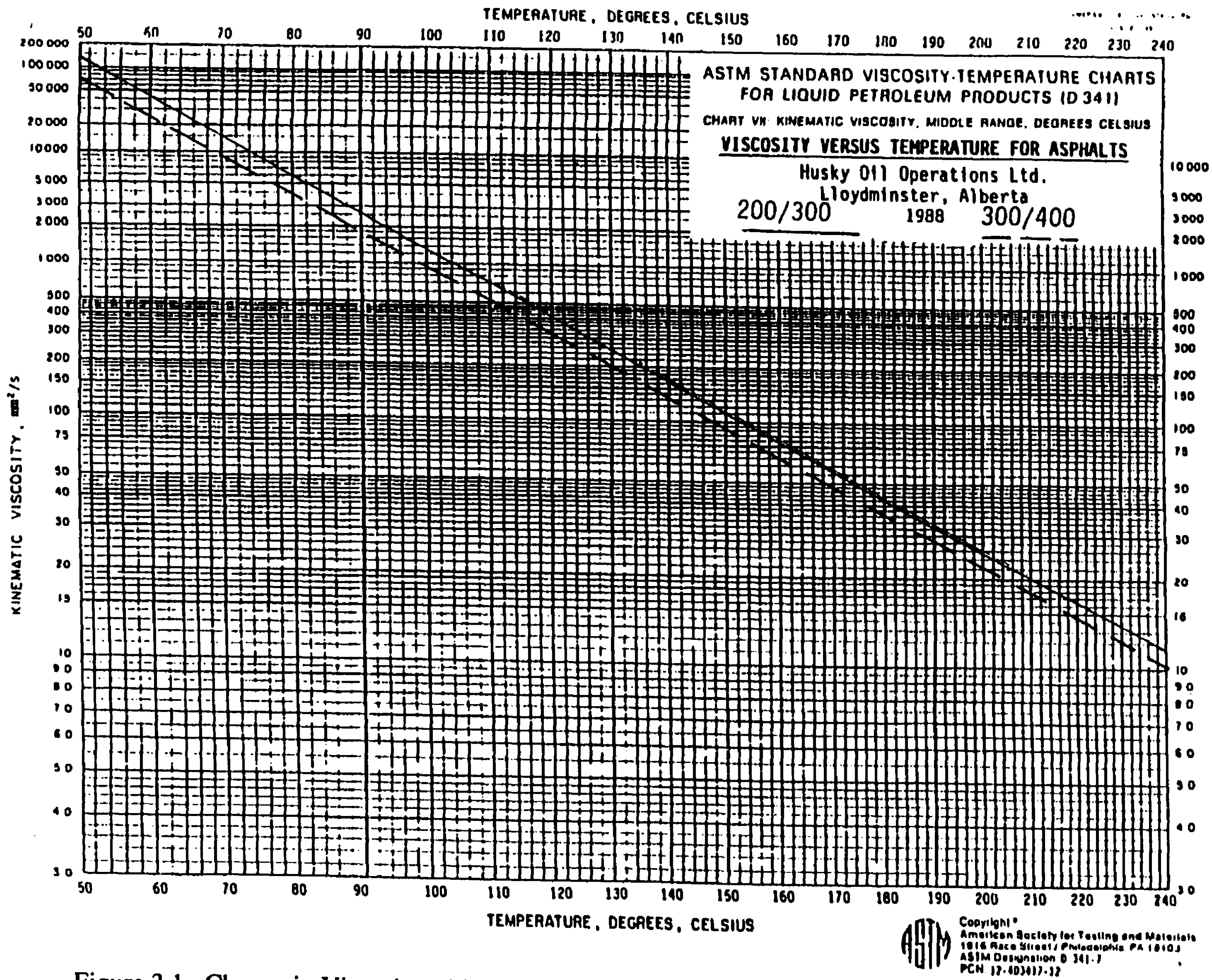


Figure 3.1 Change in Viscosity with Temperature for 200-300 and 300-400 Asphalt Cements
(Source: Husky Oil Operation Ltd.)

Table 3.3 Physical Properties of Cyclogen-L (Cyclogen)
Reported by Supplier (Witco Corporation)

Properties	Cyclogen-L
Specific Gravity	0.89 to 0.99
Boiling Point, °C	> 288
Solubility in water (by weight %)	0
Volatile (by Weight) at 25°C	< 0.01
Viscosity at 40°C, mm^2/s	18.6

Table 3.4 Physical Properties of Shell-Flex 210 (Flexon)
Reported by Supplier (Shell Canada Product Limited)

Properties	Test Method ASTM	Shell-Flex 210
Density @15°C, kg/m^3	D 1298	862.5
Viscosity @40°C, mm^2/s	D 445	20.3
Viscosity @100°C, mm^2/s	D 445	4.0
Viscosity Index	D 2270	95
Volatility, % loss	D 972	0.9
Flash Point, ° C	D 92	220
Compositional Analysis		
Aromatic	D 2007	9.30
Saturates	D 2007	90.50
Polars	D 2007	0.20

One or two kg asphalt cans were heated in an oven preheated to 130 to 140°C. The asphalt cements were heated at this temperature generally between four and five hours until sufficiently fluid to pour,. Once the asphalt cements in the cans were heated to a smooth and fluid consistency, they were poured into several quart cans of 200 cm^3 in volume. These samples were then tightly sealed, labeled, and stored at room temperature (20-30°C). The oil-based materials were fluid enough to be used without any heating at room temperature.

The preparation of blended binders and testing program were conducted at the SHRP asphalt and the main laboratories of the Manitoba Highways and Transportation, in Winnipeg, Canada. Double aging of the original asphalt, and the DSR and BBR tests on the double aged original binder, were conducted at the Asphalt Institute, Lexington, Kentucky, U. S.A.

3.3 LABORATORY AGING

For simulating the aging of the *original binder* in a Reclaimed Asphalt Pavement (RAP), the PG binder aging methods, including Rolling Thin Film Oven (RTFO) and Pressure Aging Vessel (PAV), were selected. The RTFO and PAV used in this study were manufactured by James Cox and Sons Company (U.S.A). Figures 3.2 and 3.3 show the RTFO and PAV used in this study. Details of the RTFO and PAV aging methods can be found in the AASHTO T 240 and AASHTO PP1 respectively.

The advantages of laboratory aging, compared to the extraction and recovery methods (ASTM D 2172A and D 1856), are that its preparation is less time-consuming and the aged binder is more homogenous. Although some studies (Burr et al. 1993 and Lau 1992) have shown a good correlation between properties of the PG laboratory aged and field aged asphalt cement, more research is needed to establish a general conclusion for various asphalt cements in different climate conditions.

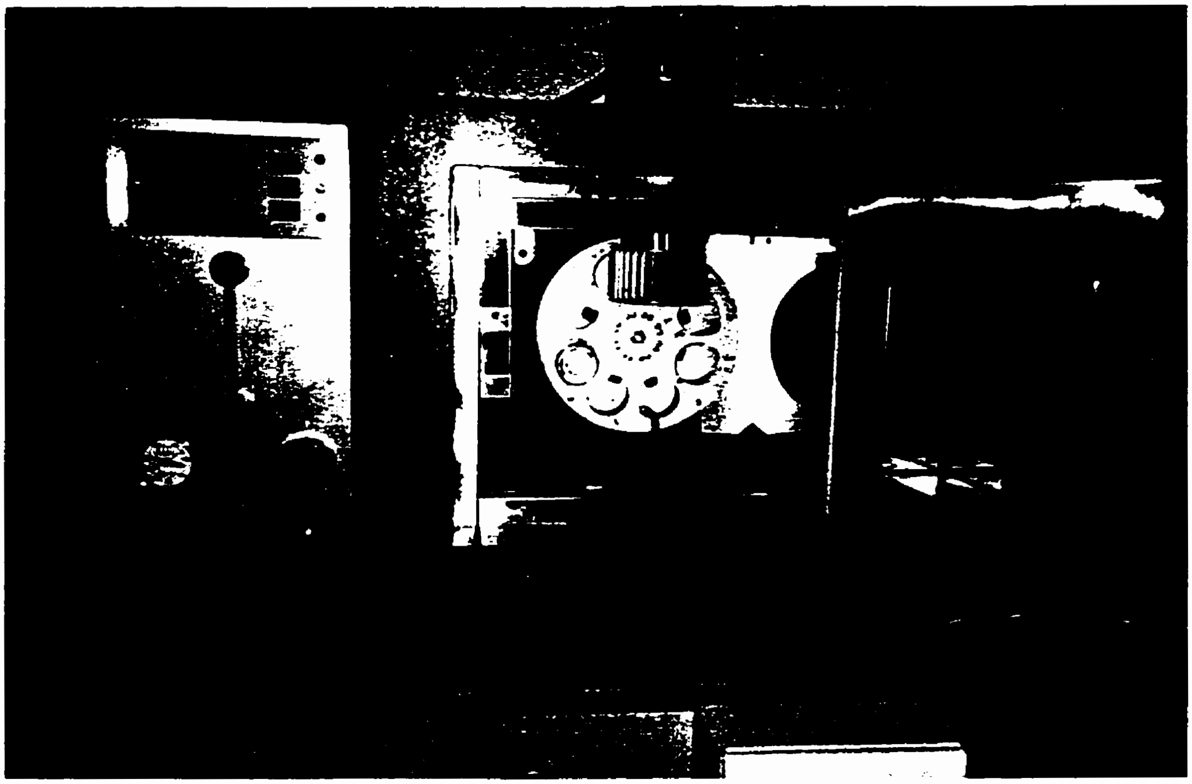


Figure 3.2 The Rolling Thin Film Oven (RTFO) Apparatus Used in this Research

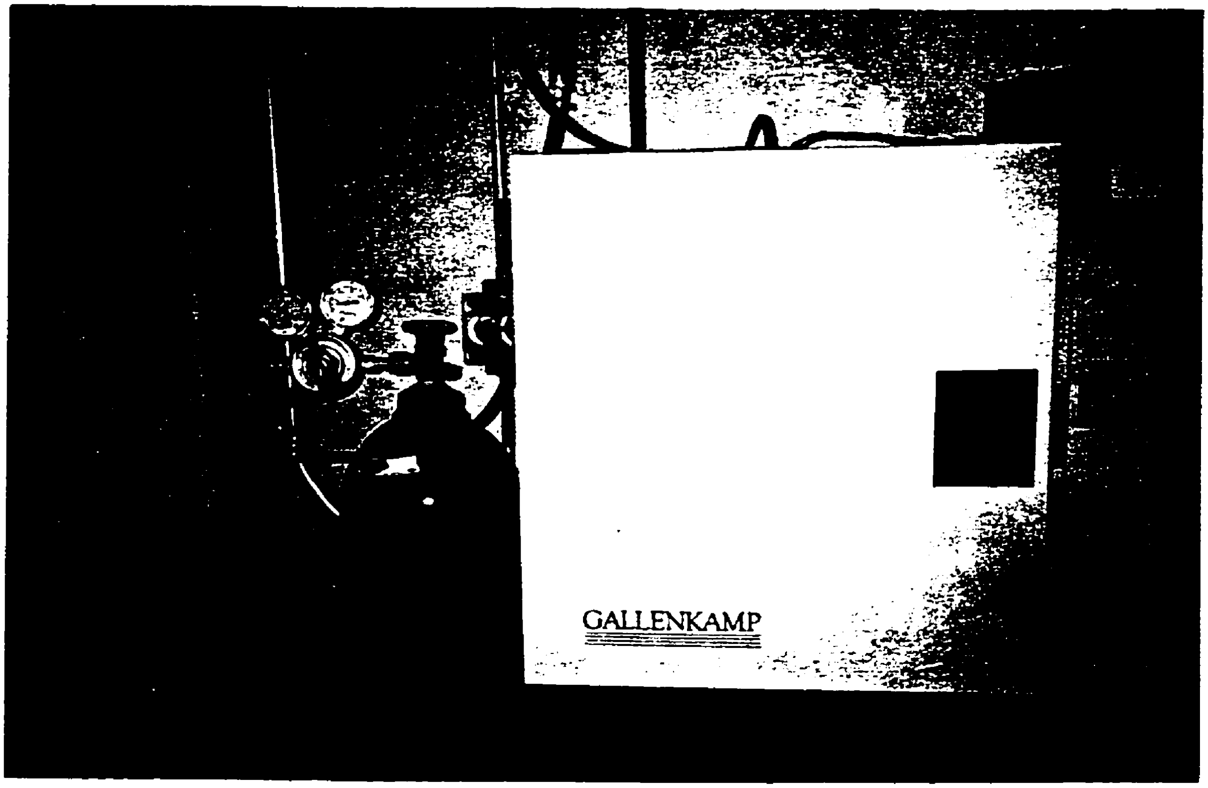


Figure 3.3 The Pressure Aging Vessel Used in this Research

The 150-200 asphalt cement was aged twice in the RTFO and PAV. Double aging, with the RTFO and PAV, was necessary to characterize the blended binders at intermediate and low pavement temperatures for fatigue and low-temperature cracks. All asphalt cements and blended binders, except the Cyclogen and Flexon were aged in the RTFO and percentages of mass losses, the weight loss before and after aging in the RTFO, were determined for all blended binders. Table 3.5 shows the mass losses for all asphalt cements and blended binders used in this study. The PG binder system suggests a maximum of one percent of mass loss for all asphalt cements (AASHTO MP 1, 1994). Therefore, none of the binders used in this study was acceptable except the 150-200. This could be related to a conservative selection of this criterion in the PG binder system, especially when it is being used for blended binders, which have more volatility content than normal asphalt cements.

Table 3.5 Mass Losses for Asphalt Cements and Blended Binders

Binders and Blended Binders	**Mass Loss % After RTFO**
150-200	0.77
200-300	1.10
300-400	1.13
30% 150-200 PAV + 70% 200-300	1.39
50% 150-200 PAV +50% 200-300	1.01
30% 150-200 PAV +70% 300-400	1.02
50% 150-200 PAV + 50% 300-400	1.40
95% 150-200 PAV +5% Cyclogen	1.10
85% 150-200 PAV + 15% Cyclogen	1.30
70% 150-200 PAV + 30% Cyclogen	1.65
95% 150-200 PAV + 5% Flexon	1.50
90% 150-200 PAV + 10% Flexon	2.60
85% 150-200 PAV + 15% Flexon	2.90

After conducting the DSR test on the virgin asphalt cements, the temperatures that satisfied the rutting criterion, $G^*/\sin\delta \geq 1$kPa, were determined. In the PG binder system, the temperature for PAV conditioning is determined based on high temperature grades. Therefore, the PAV conditioning temperatures were determined equal to 90°C for 300-400 and 200-300 asphalt cements, and 100°C for 150-200 asphalt cement. The same procedure was used to determine the PAV conditioning temperatures of blended binders. The binders were conditioned inside the PAV vessel at these temperatures and 2070 kPa pressure for 20 hours to simulate the field aging on the samples.

For preparing enough laboratory-aged binder the *original binder*, 150-200, was aged in the RTFO. Each time, after running the RTFO, around 250-260 grams of residue were obtained. After collecting enough RTFO residue, samples were aged in the PAV with 10 pans each containing 50 grams. Each time, around 400-450 grams of residue was obtained from the PAV. These aged original asphalts were kept in cans and the process was repeated several times in order to have enough residues for the blending program.

3.4 BLENDING PROGRAM

Two blending ratios, 30 and 50 percentage by weight, were used For blending, the soft asphalts with the laboratory aged *original binder*. Each of the recycling agents, Cyclogen and Flexon, were blended with the laboratory aged *original binder* in at least three blending ratios starting from 5 to 30 percentage by weight. The only criterion for selection of blending rations was that the resulted blended binder be in a range of viscosity that can perform the PG tests on them. In total 10 different blended binders were used in this study. Table 3.6 shows blending ratios (percentage weight of blending) for all blended binders used in this research.

Table 3.6 Proportion of Aged Asphalt Cement and Recycling Agents

(% by Mass)

Blended Binders	Blending Ratios (% by mass)
N1	30% 150-200 PAV + 70% 200-300
N2	50% 150-200 PAV +50% 200-300
M1	30% 150-200 PAV +70% 300-400
M2	50% 150-200 PAV + 50% 300-400
C1	95% 150-200 PAV +5% Cyclogen
C2	85% 150-200 PAV + 15% Cyclogen
C3	70% 150-200 PAV + 30% Cyclogen
F1	95% 150-200 PAV + 5% Flexon
F2	90% 150-200 PAV + 10% Flexon
F3	85% 150-200 PAV + 15% Flexon

3.5 CONSISTENCY TESTING PROGRAM

Consistency tests including the penetration at 25°C (ASTM D 5), the absolute viscosity at 60°C (ASTM D 2171), and the kinematic viscosity at 135°C (ASTM D 2170), were performed on virgin asphalt cements and blended binders. The results of these tests are presented in Table 3.7.

Table 3.7 Consistency Test Results for Asphalt Cements and Blended Binders

Asphalt Cements	Kinematic Viscosity (cP) @135°C	Absolute Viscosity (P) @60°C	Penetration 0.1 mm @25°C
150-200	380	1729	174
200-300	209	471	252
300-400	173	309	365
N1	543	4395	67
N2	411	2283	92
M1	520	4139	64
M2	396	2202	90
C1	654	7546	50
C2	312	1665	100
C3	125	290	276
F1	557	5549	63
F2	544	5404	65
F3	199	737	176

The rotational viscosity (ASTM D 4402), which has been selected by the PG binder grading system as a viscosity test, was measured on aged and virgin asphalt cements and on blended binders. The aging indices, the ratio of rotational viscosity at 135°C after aging to viscosity at 135°C before aging, were calculated. The results are given in Table 3.8. The lowest and highest aging indices were for the C3 (1.95) and F3 (9.02) blended binders. The values show that all the blended binders have similar aging indices except for the F2 and F3, which have indices two or three times more than the others. This could be related to high volatility of Flexon when exposed in the RTFO.

Table 3.8 Aging Indexes for Asphalt Cements and Blended Binders

Asphalt Cements	Rotational Viscosity(cP) Unaged	Rotational Viscosity (cP) Aged	Aging Index
150-200	250	1025	4.1
200-300	187.5	525	2.8
300-400	162.5	450	2.77
N1	637.5	2250	3.53
N2	412.5	1550	3.75
M1	450	1288	2.86
M2	337.5	987.5	2.92
C1	642.5	2375	3.69
C2	332.5	850	2.55
C3	125	244	1.95
F1	437.5	1263	2.88
F2	131	1182	9.02
F3	112.5	962.5	8.55

3.6 PG TESTING PROGRAM

The Dynamic Shear Rheometer (DSR) and the Bending Beam Rheometer (BBR) tests were performed on virgin and aged asphalt cements and blended binders. It was not possible to test the Cyclogen and Flexon with DSR because of low viscosity of these materials at room temperature. Because of unavailability of calibrated direct tension test apparatus, no Direct Tension Tests (DTT) were performed in this study. The PG parameters for DSR and BBR have shown before in Figures 2.6 and 2.7.

3.6.1 Dynamic Shear Rheometer (DSR)

As a dynamic (cyclic) testing method, the DSR applies a sinusoidal load or deformation on the asphalt cement sample and the resulting response; deformation or load is measured. The complex shear modulus, G* of the asphalt cement can then calculated. Because of the linear viscoelastic behavior the response of asphalt cement, strain or stress, is also in sinusoidal form with a delay which is called phase angle, δ. Based on the PG specification the target strain values are 12, 10, and 1 percent for virgin, RTFO, and PAV asphalt cements respectively.

The DSR has three components: loading system, temperature controller, and data acquisition system. The loading system applies a sinusoidal stress to the testing specimen and the resulting strain is measured. The water is circulated through a temperature controller that precisely adjusts (±0.1°C) and maintains the sample temperature uniformly at the desired values. The computer control (with software) and the data acquisition acquire the stress and strain and calculate the complex shear modulus and phase angle.

Figure 3.4 shows the DSR apparatus used in this research. It was manufactured by the Bohlin Instruments (U.S.A). Asphalt cements and blended binders were heated until fluid enough to pour. The heated binders were poured into a rubber mold and allowed to cool. The binder samples were transferred to the parallel plate of the DSR and the plate gaps were adjusted to the DSR. The samples were trimmed and after reaching

Figure 3.4 The Dynamic Shear Rheometer Apparatus Used in This Research

the desired temperatures were tested. All binders, virgin and aged, were tested at different temperatures: 7, 13, 19, 25, 31, 37, 46, 52, 58, 64, and 70°C. Two different geometries of parallel plates were used. For temperatures less than 40°C, the 8-mm diameter plate with 2-mm plate gap was used, and for temperatures more than 40°C, the 25-mm plate with 1-mm gap was used.

There were some difficulties in using the 8-mm plate at very low temperatures such as 7°C and the 25-mm plate at very high temperatures such as 70°C. The problems could be related to the high and low level of induced stresses for a constant strain at low and high testing temperatures. The testing sample for each of the geometry plates did not change and tests were performed on the same sample after reaching the next desired temperature. The complex shear modulus (G*) and the phase angle (δ) were obtained at a frequency of 10 rad/s. (equal to about 1.59 Hz) based on AASHTO TP5. A typical DSR output is shown in Figure 3.5.

3.6.2 Bending Beam Rheometer (BBR)

The BBR is a creep test that measures the stiffness (S) of asphalt cement on a beam sample on a controlled low-temperature condition. The other BBR measured parameter is the m-value, which is the slope of the log of S verses log of loading times at 60 seconds loading time.

The essential elements of the BBR are a loading frame, controlled temperature fluid bath, computer control and data acquisition system, and test specimen. The controlled temperature bath contains a fluid with very low freezing temperature such as ethylene glycol, methanol, and water. This fluid is circulated between the testing bath and a circulating bath, which controls the fluid temperature to within ±0.1°C. The data acquisition system consists of a computer (with software) connected to the BBR to control test parameters and acquire load and deflection results.

DSR Project - SHRP Binder Test - Strain Controlled
(Created by Version 2.04)

Parameters:

Measurement Type:	Intermediate Temperature Range
Target Temperature:	6.5 °C
Strain Amplitude:	1.00 percent
Plate Diameter:	8.0 mm
Plate Gap:	1.5 mm
Equilibration Time:	3.0 minutes

Ancillary Info:

Operator ID:	Hamid
Sample ID:	m1pav
Sample Type:	Pressure Aging Vessel Residue
Test Number:	0036

Measurement Results:

Completed:	6/9/95 5:28:57 PM
Modulus (G*):	2.3311E7 Pascal
Phase Angle (delta):	34.7 degrees
G*•sin(delta):	1.3257E7 Pascal
Strain Amplitude:	0.42 percent
Final Temperature:	6.6 °C
Osc. Frequency:	10.08 radians/second
Test Status:	FAILED

Figure 3.5 Typical Output of DSR Test

There were some difficulties in testing the BBR at high temperatures such as -6°C for some of the blended binders. This problem was related to the high deflection of the soft samples at -6°C temperature. Therefore, blended binders were tested at -18, -24, and -30°C. The stiffness (S) and the m-values were obtained at 60 seconds loading times for all asphalt cements and blended binders. All the BBR tests were conducted on the aged binders. It was not possible to test the two recycling agents, Cyclogen and Flexon, with the BBR due to the low viscosity and difficulties of conditioning in the RTFO and PAV. The BBR used in this study was manufactured by the Canon Instrument Company (U.S.A.). Figure 3.6 shows the BBR apparatus used in this research. Figure 3.7 depicts a typical output of the BBR test.

3.7 PRECISION OF THE PG TESTING RESULTS

The PG binder system was introduced in 1993. Therefore, compared to the traditional binder tests there is limited experience with this new system. One of the important concerns of the PG binder system is the repeatability and variation of PG binder test results.

Christensen (1992) reported the precision of the DSR in the form of standard deviations or coefficient of variation in repeated measurements. The average of the Coefficient of Variation (CV) for various ranges of the complex shear modulus was 14.7 percent with a maximum of 23.7 percent for G* between 30,000 and 100,000 Pa. The average of standard deviation for the phase angle was 0.59 degree with a maximum of 0.94 degree related to complex shear modulus between 10,000 and 30,000 kPa.

A recent repeatability testing program conducted by the AASHTO (1996) Materials Reference Laboratory (AMRL) reported that the between-laboratory standard deviation for the determination of phase angle for original asphalts as about 0.35 degrees for DSR results conducted at 64°C with 25-mm plates and a 1-mm gap.

Figure 3.6 The Bending Beam Rheometer Used in This Research

TEST INFORMATION

Project : recycling
Operator: Hamid
Specimen: N1
Time : 05:21:07
Date : 06/22/95
File : 0622951

Target Temp: -30.0 C
Actual Temp: -31.2 C
Soak time : 60.0 min
Beam Width : 12.70 mm
Thickness : 6.35 mm

Conf. Test: 2.00332e+00
Date: 06/22/95
Load Const: 0.2457
Defl Const: 0.002491
Date: 06/22/95

RESULTS

t Time (sec)	P Force (N)	d Defl (mm)	Measured Stiffness (kPa)	Estimated Stiffness (kPa)	Difference (%)	m-value
8	1.017	.08130	1009000.0	1013000.0	.4202	.212
15	1.017	.09196	892000.0	885400.0	-.7441	.216
30	1.017	.1080	759500.0	760800.0	.1756	.221
60	1.018	.1259	651900.0	651500.0	-.06384	.226
120	1.019	.1486	553100.0	556000.0	.5330	.231
240	1.027	.1746	474400.0	472900.0	-.3150	.236

Regression Coefficients

a = 6.190 b = -.1970 c = -.008227 R^2 = 0.999720

- CANNON BENDING BEAM RHEOMETER - V3.1 P to print - ESC to continue

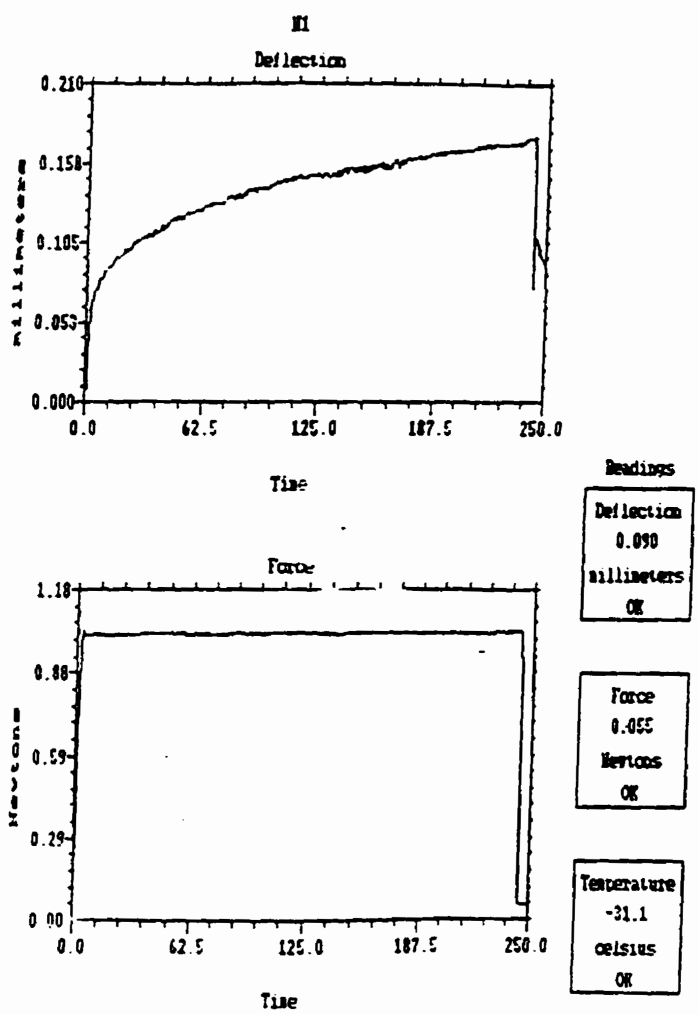


Figure 3.7 Typical Output of BBR test

In the AASHTO study, the standard deviation increased slightly to 1.4 degrees when testing aged (PAV) asphalt cements at 25°C with 8-mm diameter plate and a 2-mm gap. Using these statistics, individual test results reported by different laboratories should not differ by more than 0.85 degrees for virgin binders tested at warmer temperatures. This difference increased to 3.96 degrees for aged binders at colder temperatures. Some other results from this study for PG criteria are given in Table 3.9.

Bahia et al. (1992) reported the repeatability of BBR test results for two replications of eight core asphalts based on standard deviation and coefficient of variation. The results are summarized in Table 3.10. Based on the result of this study, the coefficients of variation depend upon the stiffness levels. As the stiffness increases the coefficient of variation increases. This could be related to the smaller deflection for higher stiffness asphalt cements and consequently more experimental "noise" for these levels of results.

3.7.1 Repeatability of the DSR Test

A small-scale repeated test program was analyzed to have some idea of how much the laboratory results in this study were accurate. For this purpose, the complex shear modulus and phase angle for six different replications of the virgin, and four replications of the aged 150-200 asphalt cement were measured on various samples at different times. The coefficients of variation were obtained for each temperature by calculating the average of the sample means and standard deviations for the replicates.

Table 3.11 summarizes the average coefficients of variation for the complex shear modulus and phase angle of virgin and aged 150-200 asphalt cement, for all temperatures.

Table 3.9 Estimated PG Temperature Variability from SUPERPAVE Test Variability (Based on 1996 AASHTO Round Robin Results)

Number of Labs	**PG Criteria**	**Average Value**	**Standard Deviation**	**Change in Property per °C**	**Estimated Temperature Reproducibility °C**
67	G*/sin δ (unaged)	1.354 (kPa)	0.14	0.09	4.3
68	G*/sin δ (RTFO)	3.297 (kPa)	0.46	0.22	5.8
60	G* sin δ (PAV)	2253 (kPa)	504	530	2.6
55	Stiffness (PAV)	183 (MPa)	20.7	38	1.5
59	m-value (PAV)	0.344	0.018	0.01	5.0

Table 3.10 Statistical Summary of Stiffness Repeatability for BBR (Bahia et al. 1992)

Stiffness (MPa)	**Mean (MPa)**	**Standard Deviation**	**Coefficient of Variation %**
<124	71	1.16	1.6
124-320	211	5.47	2.6
320-620	457	13.69	3.0
>620	1000	37.78	3.8

Table 3.11 Summary of Coefficients of Variation of Complex Shear Modulus and Phase Angle for 150-200 Asphalt Cement

DSR Parameters	**G***		**δ**	
Conditioning	**Virgin**	**Aged**	**Virgin**	**Aged**
Coefficients of Variation, %	31.1	32.9	2.29	4.54

Based on the results presented in Table 3.11 it can be concluded that the average CV of the complex shear modulus was several times more than the average CV of the phase angle. The average CV of the aged and virgin complex shear modulus were very close to each other, but the average CV of the aged phase angle was more than the phase angle of unaged asphalt cement. It could be concluded that the phase angle has a better repeatability than the complex shear modulus. There was no special trend in the effect of temperatures on CVs. In most cases, CVs changed significantly when the temperature changed from 37° to 46°C. This could be attributed to change in the binder sample and change in the geometry of parallel plate of the DSR.

The primary sources of error in the DSR analysis are listed below:

1. Error in temperature control--The complex shear modulus of asphalt cement will typically change from 10 to 25 percent per degree Celsius. Therefore, differences between the actual average specimen temperature and the recorded or specified temperature, and the presence of temperature gradients within the specimen might be sources of errors. The dynamic shear rheometer used in this study had control temperature accuracy within ± 0.1°C.

2- ***Errors in sample trimming***--Because the complex modulus is proportional to the fourth power of the radius of asphalt sample, errors in trimming of the samples can be another important source of errors in the DSR testing.

3-***Errors in setting and recording the specimen gap***--The complex modulus changes inversely with the gap or sample thickness. This could be a random error basis or the result of thermal expansion of the tools and/or specimen with changes in temperature.

4-***Calibration error*** --Load and strain transducers and temperature control should be calibrated frequently. Overall calibration with a reference fluid is also recommended. Lack of calibration might be a source of errors in the DSR test results. The DSR used in this study was calibrated in the time of installation but there was not any more calibration during this testing program.

3.7.2 Repeatability of the BBR Test

To test the precision of the BBR, four different replications of the stiffness for eight different blended binders were measured at three different loading times (at –18°C). The coefficients of variation were obtained by calculating the average of the sample means and standard deviations for the replicates. The range of studied stiffness was between 404,200 and 34,590 kPa and the range of m-value was between 0.593 and 0.270. Table 3.12 summarizes the average Coefficient of Variations of stiffness at three loading times (8, 60, and 120 seconds) at -18°C for eight binders.

The maximum CV of stiffness was equal to 17.3 percent for the S1 blended binder at 240 seconds loading time. The minimum CV stiffness was equal to 0.7 for the C2 blended binder at 60 seconds loading time. The statistical analysis showed that there was no significant effect of the loading times on the repeatability of the BBR parameters. This conclusion is in agreement with study conducted by Bahia et al. (1992).

Table 3.12 Coefficient of Variations (Percent) for BBR Parameters

Loading Times (Seconds) / BBR Parameters	8	60	240
Stiffness (S)	8.9	7.7	9.9
m-Value	4.4	3.8	4.8

The primary sources of error in the BBR analysis are listed below:

1. ***Error in temperature control***--Differences between the actual specimen temperature and the recorded temperature and non-uniformity in temperature liquid bath are the main source of temperature errors in the BBR. The BBR used in this study controls temperature to within about ± 0.1°C.

2. ***Specimen preparation***--Two different types of molds have been suggested for molding the asphalt for BBR samples. The first method uses an aluminum mold and the other method uses a silicon rubber mold with a Plexiglas™ plate. All the samples used in this study have been made with aluminum molds. There is no technical report regarding the difference in BBR results by using these two different molds. Assembling the mold, especially the aluminum mold, filling the binder in the mold, and demolding need considerable attention when preparing the samples. Samples with air bubbles inside and/or on the surface can cause errors in BBR results.

3. ***Thermal history effect***--The physical hardening effect might be different for some asphalts at temperatures below about 0°C. This phenomenon can lead to changes in stiffness in some types of asphalts at low temperatures (Bahia et al. 1992).

4. ***Calibration error***--Load cell, displacement transducer, and temperature control should be calibrated frequently.

The other source of variability can be related to the difference between types of testing apparatus. For example, in the evaluation of the PG binder equipment, it was determined that there is a difference in the results reported by the Canon and AST bending beam rheometers. The Cannon rheometer reports m-values which, are typically 0.01 higher than ATS. With a constant criterion for m-value equal to 0.003 in the PG specification, this difference is important and may affect proper binder selection. The reason for this difference may be related to the way the various devices define the zero time of loading.

Table 3.13 summarizes the statistical analysis related to the repeatability of the PG binder tests from this study with some other research, mentioned before. The highest difference is related to complex shear modulus with maximum 33 percent coefficient of variation compared to 10-20 percent reported by Christensen (1992). In other cases, the coefficients of variation are comparable with other studies. The BBR has a better repeatability than the DSR.

The CVs that reported in Table 3.13 show a low repeatability for PG parameters especially for complex shear modulus. This can be a big concern for implementation of PG tests for practical use and should be solved by improving testing procedures and modifying the equipment.

Table 3.13 Comparison of Coefficients of Variation (Percent) for PG Binder Tests

	G*	δ	S	m
Christensen (1992)	9.8-18.5	0.58-5.09	NA	NA
AASHTO (1996)	NA	NA	11.31	5.23
Bahia (1992)	NA	NA	1.6-3.8	NA
This Study	31.12-32.92	2.29-4.54	8-10	4.41-4.82

In comparing different precision studies, it is important to recognize that some of these studies have been conducted in very highly controlled conditions, with a well-trained operator. In addition, it is important to recognize that statistical analysis, from this study, represents a measure of the variability for just eight aged binder samples, in one laboratory, with one operator. The variability levels reported reflect experimental error involved in not only the testing operation but also in the variation, caused by specimen preparation procedure, temperature control, and variability because of isothermal age control.

3.8 DISCUSSION OF THE RESULTS

The DSR and BBR testing results for all binders are presented in Appendix A. Each DSR result is the average of at least two repetitions on the same sample. The BBR results are the average of at least three different samples. The DSR and BBR results for double aged original binder are also presented in Appendix A. As can be seen in Tables A.1 to A.9, the binders exhibit a wide range of properties. The G* at 70°C ranges between 7770 kPa and 1021 kPa. The phase angle ranges between 46.6 and 68.5

degrees. The range in G^* is between 30132 kPa and 4821 kPa, at 7°C. The range in phase angle after PAV aging is between 29.8 and 48.5 degrees at the same temperature. The BBR results, in Tables A.10 to A 13 in Appendix A, show a wide variation in the S (60) and m-values (60). At -30°C, the range is between 712.5 MPa and 337.4 MPa for S (60), and between 0.216 and 0.316 for the m-values (60). The ranges give an indication of properties included in the analysis of this study.

Figures 3.8 to 3.16 compare the complex shear modulus and phase angles of aged and unaged asphalt cements and blended binders from the isochronal curves. Some common unique characteristics of the rheological behavior of asphalt cements and blended binders can be observed from isochronal curves. At high temperatures, asphalts behave like viscous fluids, and at very low temperatures, they behave like elastic solids. As the temperature increases, G^* decreases and δ increases. Decreases in G^* indicate that the asphalts are becoming softer and consequently less resistant to deformation, and increases in δ indicate a decrease in elasticity or ability to alter energy. At high temperatures, the δ values approached 90° for all asphalts, which indicates the approach to complete viscous behavior or complete dissipation of energy in viscous flow. As the asphalts ages, the G^* increases and the δ decreases. This means that the aged asphalts had more resistance to deformation and less viscous behavior. Therefore, the aged asphalts are more susceptible to cracking, especially at low temperatures. The difference between δ of unaged and aged asphalts decreases as the temperature increase.

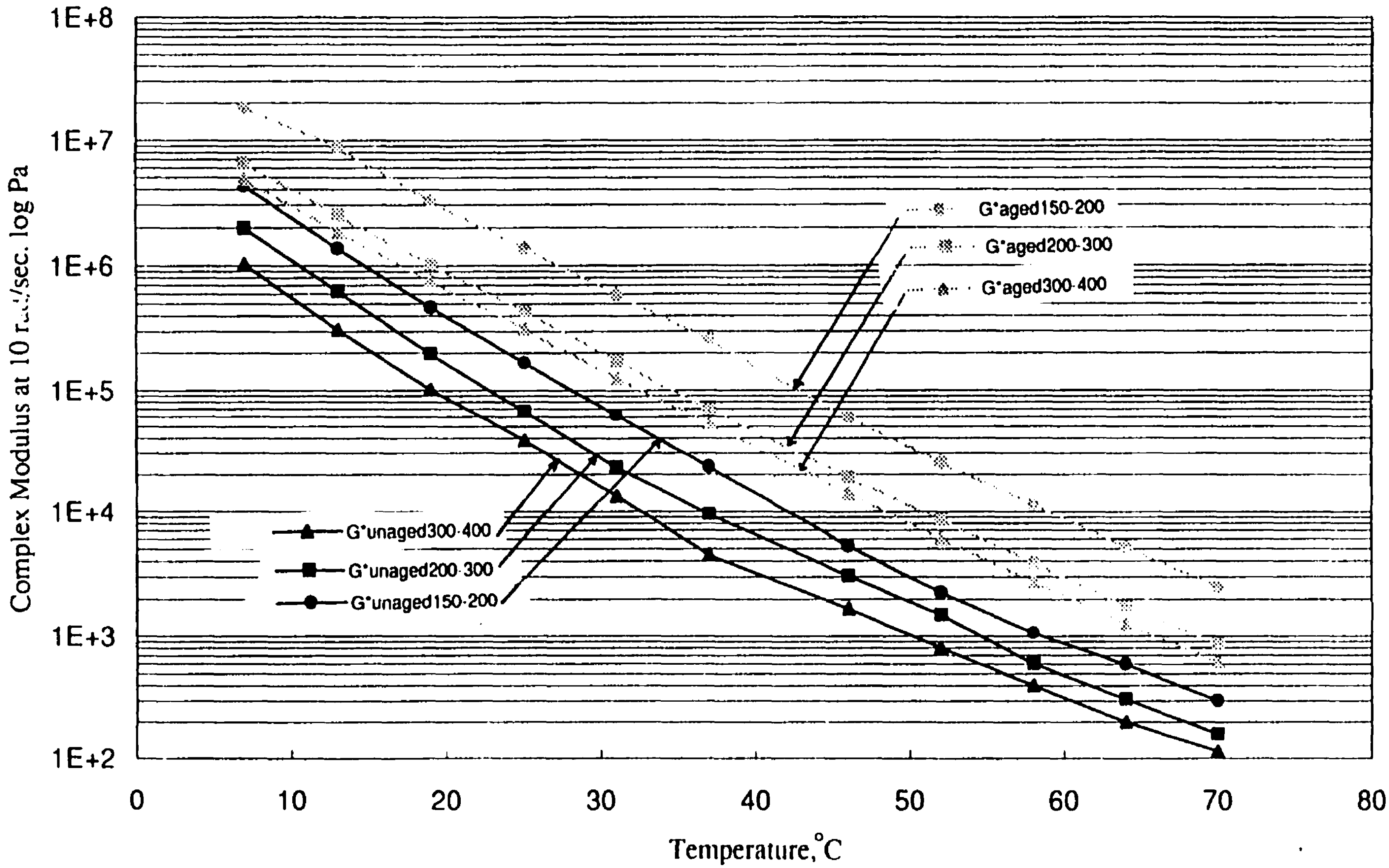


Figure 3.8 Isochronal Curves of the Complex Modulus for Aged and Unaged Asphalt Cements (150-200, 200-300, and 300-400)

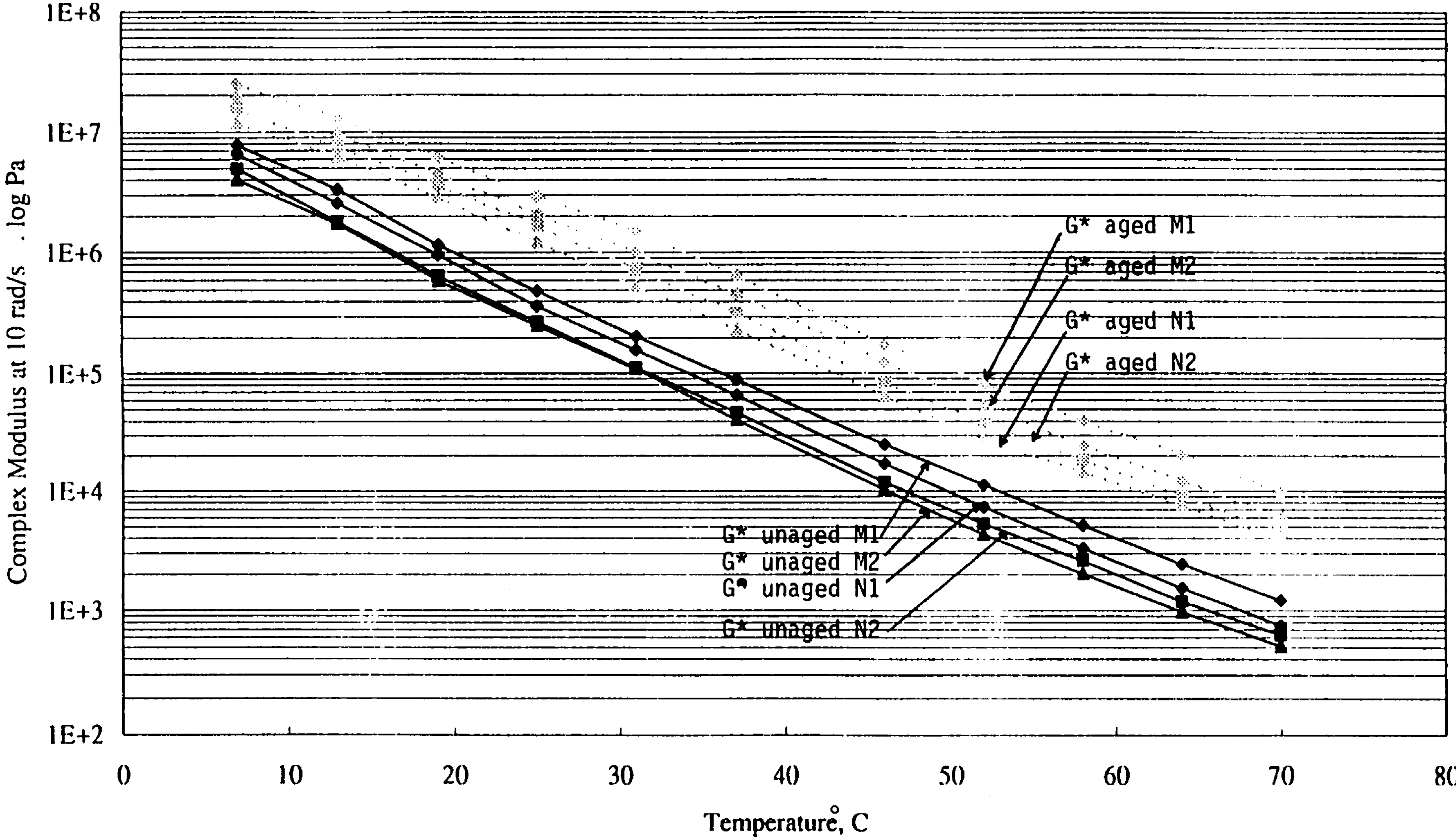


Figure 3.9 Isochronal Curves of the Complex Modulus for Aged and Unaged Binders Blended with 200-300 and 300-400 Asphalt Cements

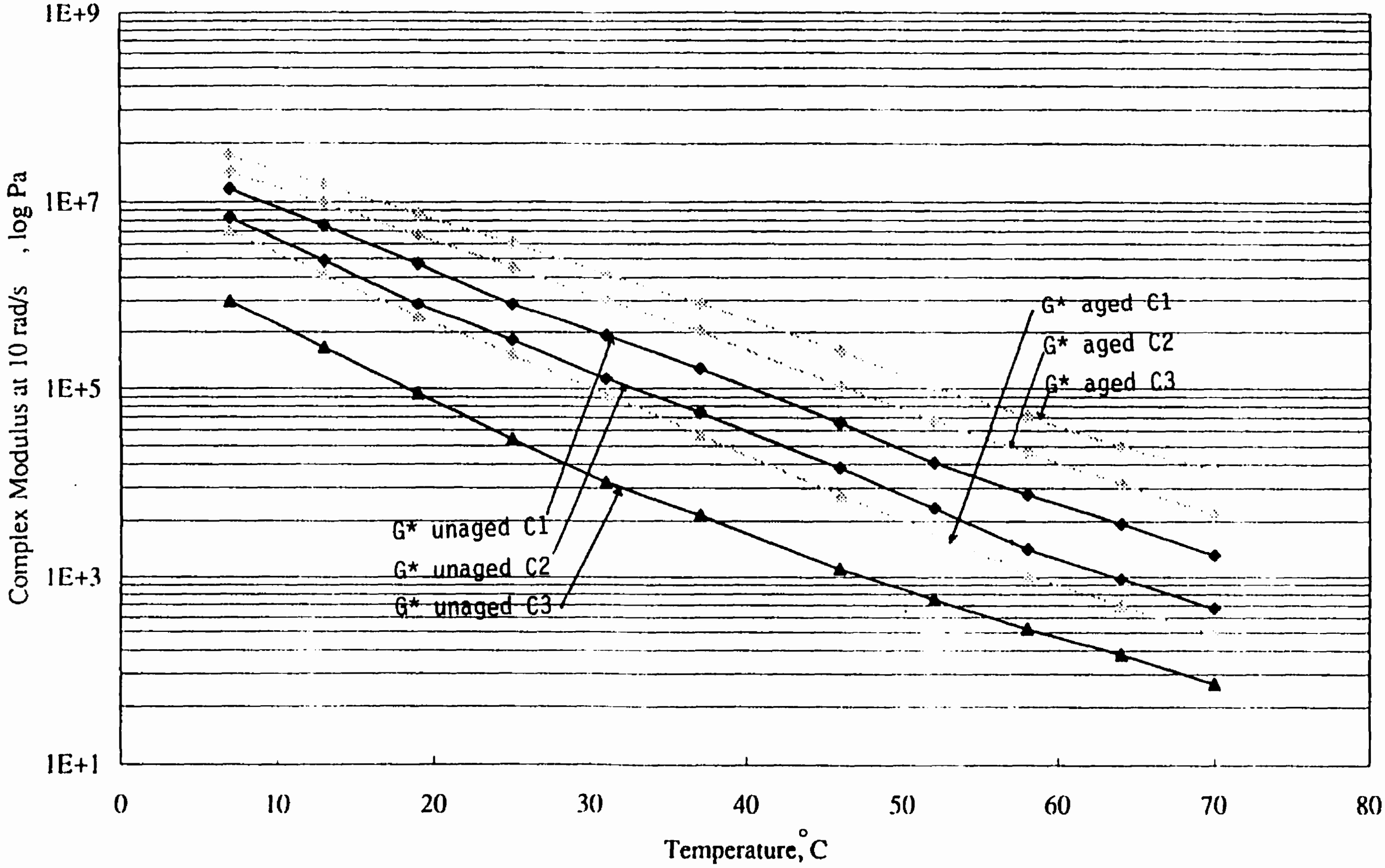


Figure 3.10 Isochronal Curves of the Complex Modulus for Aged and Unaged Binders Blended with Cyclogen

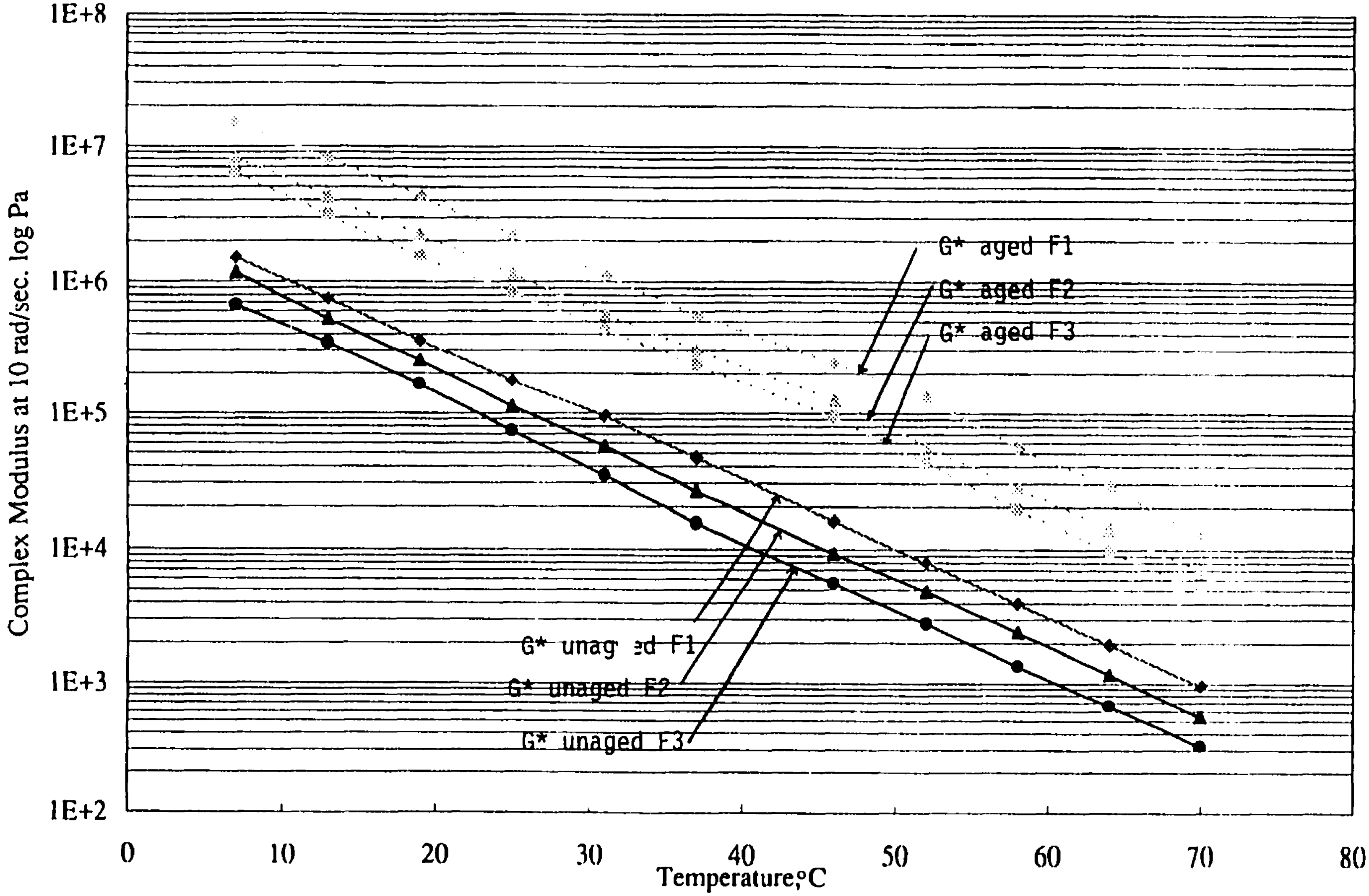


Figure 3.11 Isochronal Curves of the Complex Modulus for Aged and Unaged Binders Blended with Flexon

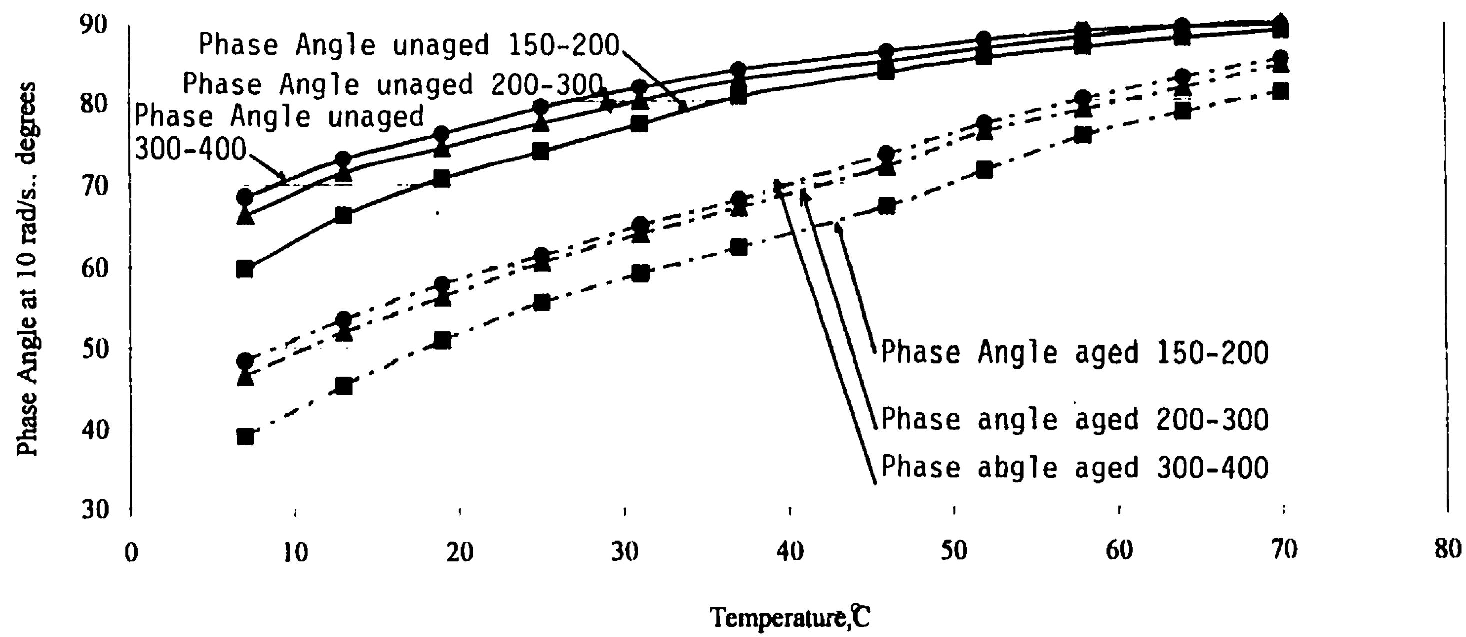
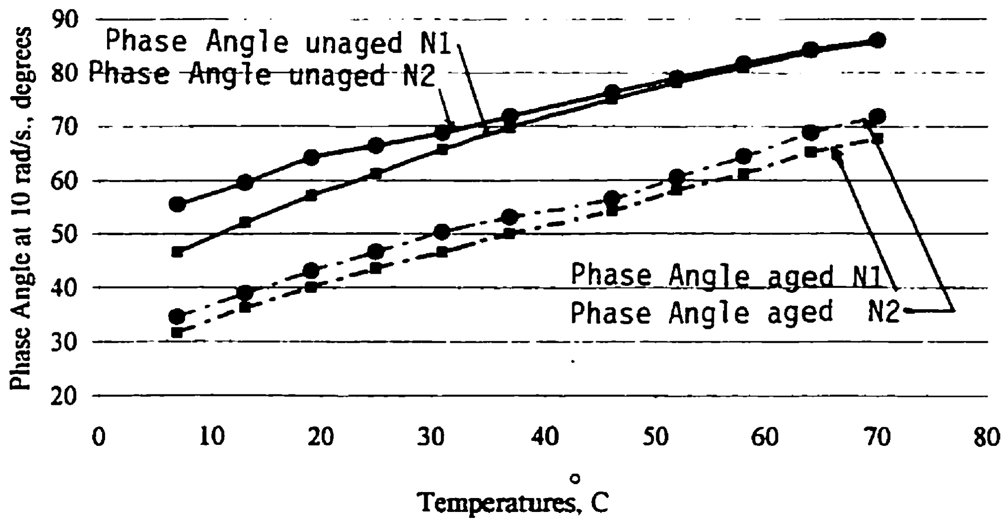


Figure 3.12 Isochronal Curves of the Phase Angle for Aged and Unaged Asphalt Cements (150-200, 200-300, and 300-400)



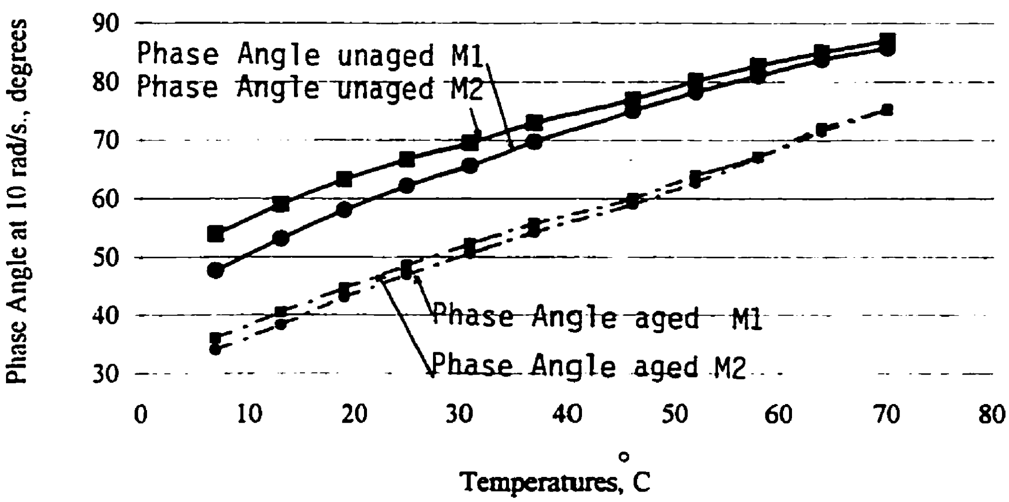


Figure 3.13 and 3.14 Isochronal Curves of the Phase Angle for Aged and Unaged Binders Blended with 200-300 and 300-400 Asphalt Cements

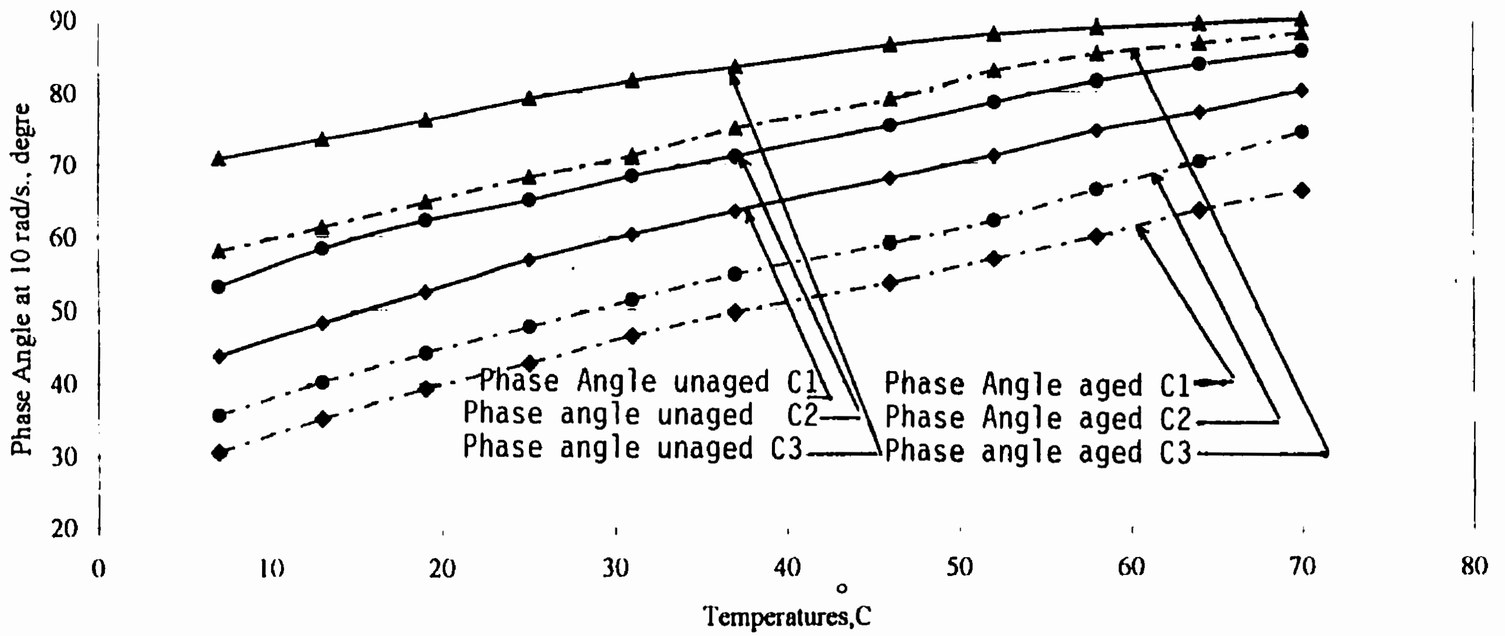


Figure 3.15 Isochronal Curves of the Phase Angle for Aged and Unaged Binders Blended with Cyclogen

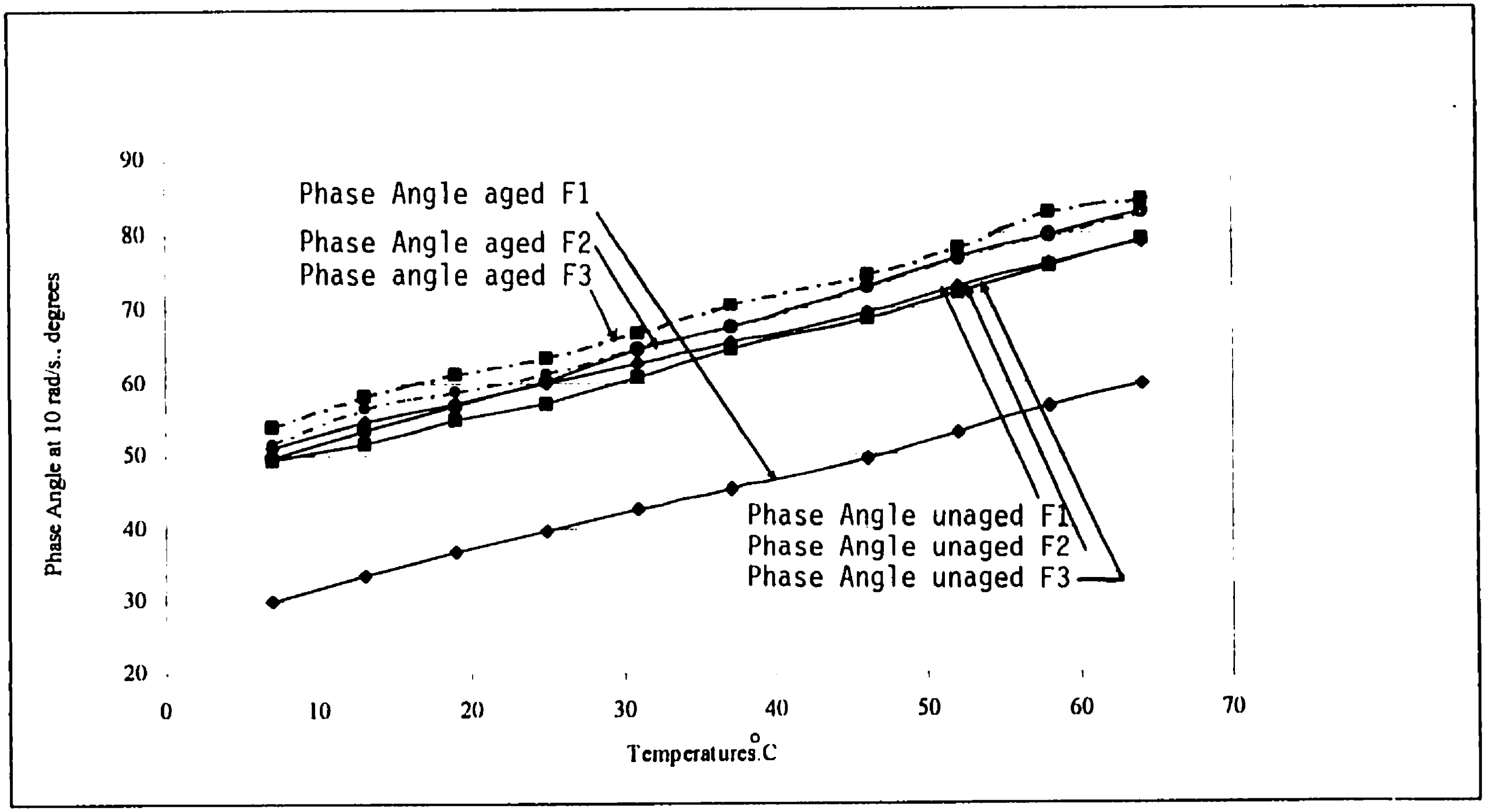


Figure 3.16 Isochronal Curves of the Phase Angle for Aged and Unaged Binders Blended with Flexon

3.9 PERFORMANCE GRADE OF BINDERS

The accuracy of PG grading of asphalt cement depends on the accuracy of PG testing results. With the expected variations, which were explained before, it is possible that the same binder be graded with different PG grades at different labs.

The asphalt cements and blended binders used in this research were graded based on the PG binder system. There were some difficulties for grading some of the blended binders. One difficulty in grading of asphalt cement with the PG binder system is the question of how much tolerance in the value of PG criteria is accepted for shifting between criteria temperatures. For example, the $G^*/\sin\delta$ for C2 blended binder was 946 and 2012 Pa at 64° and 58°C respectively. Because the value of $G^*/\sin\delta$ at 58°C was very close to the PG specification, $G^*/\sin\delta$ <1000 Pa, it was far from the value at 64° C, it was decided this blended binder be graded at 64. For F2 blended binder, the same parameter was 1174 Pa at 64 °C. With a minimum of 1000 kPa for this parameter in the PG binder system, both binders were graded at 64. This means that although there was more than 20 percent difference between value of $G^*/\sin\delta$ both binders are graded at the same group. The other difficulty was incompatibility of temperatures for different parameters. This means that, based on the PG binder system, the S ≤300 MPa and m-value ≥0.300 should be satisfied at the same temperatures but this may not have happened in some cases. For example, for N2 blended binder the S value was satisfied at –18°C but the m-value was satisfied at –24°C. In these cases the more conservative temperature, equal –18°C, was selected.

Some of these issues were reported recently by some researchers. Puzic et al. (1997) reported the PG temperature variability as measured with four operators within the Imperial Oil Lab. The results are presented in Table 3.14. Puzic et al. concluded that the PG binder system is able to differentiate well between three asphalts graded on the

Canadian General Specification Board (CGSB). They showed that high inter-laboratory measurement variability causing very high variability in estimating maximum and minimum PG temperatures could mean a potential lab-to-lab difference in performance grading of 1-2 grades. This means that the differences in performance grading between labs could be a result of the properties of the binder being very close to the boundaries or located at the corner of a "PG box".

Table 3.14 The PG Temperature Variability as Measured Within Imperial Oil Lab (Four Operators from April to Augest, 1996)

Number of Observations	**PG Criteria**	**Average Value °C**	**Standard Deviation**	**Estimated PG Temperature Precision, °C**
10	G*/sin δ (unaged)	58.6	0.500	1.39
10	G*/sin δ (RTFO)	59.6	0.596	1.65
9	G*. sin δ (PAV)	17.5	0.495	1.37
10	Stiffness (PAV)	-19.6	0.195	0.54
10	m-value (PAV)	-18.8	0.546	1.51

Table 3.15 shows the PG of all the asphalts and blended binders used in this study. With blending of the softer asphalts, it can be seen that the PG grades changed from PG 64-34 to PG 70-28. None of blends (30% and 50%) could meet the (-40) grade for low-temperature.

3.10 CHAPTER SUMMARY

The physical properties of the materials and testing equipment used in this research were explained. The original asphalt cement was aged with the RTFO and PAV. The laboratory aged binder was blended with two soft asphalt cements and two recycling agents with different proportions (percentage of mass) to have 10 different blended binders. Blended binders were tested with the DSR and BBR at a wide range of

Table 3.15 The Performance Graded (PG) for All Binders Used in This Study

Binders	SHRP PG
150-200	**PG 58-28**
200-300	**PG 52-34**
300-400	**PG 46-34**
M1 (30% of 300-400+70% of PAV 150-200)	**PG 64-34**
M2 (50% of 300-400+50% of PAV 150-200)	**PG 64-34**
N1 (30% of 200-300+70% of PAV 150-200)	**PG 70- 28**
N2 (50% of 200-300+50% of PAV 150-200)	**PG 64-28**
C1 (5% of Cyclogen +95% of PAV 150-200)	**PG 70-28**
C2 (15% of Cyclogen +85% of PAV 150-200)	**PG 64-28**
C3 (30% of Cyclogen +70% PAV 150-200)	**PG 46-34**
F1 (5% of Flexon +95% of PAV 150-200)	**PG 64-34**
F2 (10% of Flexon +90% of PAV 150-200)	**PG 64-34**
F3 (15% of Flexon +85% of PAV 150-200)	**PG 58-34**

temperatures from –30° to +70°C. The repeatability of the PG binder testing results was compared with some other studies. Based on limited number of tested materials and studying the repeatability of obtained data, the following conclusions are drawn from testing results:

1. The mass loss criteria selected by the PG asphalt system appears conservative, especially for blended binders, which have higher volatile contents.
2. Temperature control, sample trimming, setting and recording the gap and calibrated error may be some of the important sources of error in the DSR analysis results.
3. Temperature control, specimen preparation, thermal history effect, and calibration errors are considered to be the most important sources of errors in the BBR analysis results.

4. Phase angle has a better repeatability than the complex shear modulus in the DST test. BBR equipment has a better repeatability than the DSR equipment.
5. The precision of PG testing results in this study is acceptable when compared with other studies. The complex modulus results had 10 percent more deviation compared to other studies. Generally the repeatability of the PG testing results is low especially for complex shear modulus and needs improvement in testing procedures and equipment.

CHAPTER FOUR

MODELS FOR BLENDED BINDERS

4.1 INTRODUCTION

In order to effectively predict responses of the recycled asphalt pavement due to different traffic loads and environmental conditions, the blending of the aged asphalt cement and recycling agent should be characterized over a wide range of temperatures and loading times. Performance models for blended binder can lead to better selection of recycling agents in the recycled asphalt mixture and consequently achieve a better recycled pavement performance. This is very important for cold climate conditions like those in Canada and the northern United States, where the asphalt cement is the dominant component in performance of asphalt pavements.

The main purpose of this chapter is to describe the models developed for blended binders. These models are based on rheological properties of asphalt cement such as the complex shear modulus (G^*), the stiffness (S), the phase angle (δ), and the m-value of blended binders. The hypothesis used for building models in this dissertation was that: *a linear model ca predict the change in PG parameters with blending ratios*. Based on developed models, two procedures were evaluated for selecting recycling agent in recycled mixtures. The performances of blended binders were predicted based on the PG binder specification. Finally, as a case study, the performances of blended binders were compared for use in Saskatoon, Saskatchewan area climate conditions.

4.2 MODELS FOR BLENDED BINDERS

There are some difficulties in characterizing viscoelastic materials such as asphalt cements. One difficulty is that properties of asphalt cements change over time (aging), and these changes might be different for various types of asphalt cement. Another difficulty is that the behavior of asphalt cements should be characterized over a wide range of temperatures and loading times or frequencies. Finally, characterization of asphalt cement should address pavement distresses directly. The PG-binder system, developed by the SHRP, considers these requirements with cyclic and creep tests over a wide range of temperatures and loading times on virgin and laboratory aged asphalt cements. This new asphalt cement specification relates the test results directly to the main asphalt pavement distresses.

Recycling is an economical and environmentally friendly method for rehabilitation and maintenance of asphalt pavements. In most asphalt pavement recycling projects, it is necessary to add softer asphalt and/or recycling agent to restore the aged asphalt cement properties. Selection of the type and amount of recycling agents are critical steps in the design of reclaimed asphalt mixtures. The current methods for selection of recycling agent are based on traditional testing procedures such as viscosity and penetration. As discussed in the literature review, there are several shortcomings in traditional asphalt cement specifications and these specifications do not explain the asphalt cement behavior in all ranges of temperature and loading time.

Irving (1977) reviewed more than 50 different linear and non-linear equations for blended liquids and concluded that the best descriptive equation, which has the widest applicability and is comparatively simple, is the Grunberg equation [4.1]:

$$ln\eta = x_1\, ln\eta_1 + x_2\, ln\eta_2 + x_1 x_2 G_{12} \qquad [4.1]$$

where:

G_{12} is usually a constant and x_i may be mole, mass, or volume fraction and η_i are viscosity of the liquids.

A special case of this equation with $G_{12} = 0$, called the Arrhenius Equation, has been used widely for the prediction of viscosity of blended asphalt cements or a blended asphalt cement/recycling agent (oil based materials).

$$ln\eta = x_1 \, ln\eta_1 + x_2 \, ln\eta_2 \qquad [4.2]$$

Equation [4.2] represents a linear relationship between $ln\ \eta_1$ and $ln\ \eta_2$ with x_1 or x_2 as coefficients.

The main objective of this research was developing models for blended binders based on rheological parameters. The hypothesis used for this purpose is that: *a linear relationship exists for the characterization of rheological measurements (G^*, S, δ, and m-value) of blended binders with proportion of the recycling agent.* The relationship should be tested for both virgin and aged blended binders and for a wide range of temperature and loading time. If there is a linear relationship for change in rheological parameters of blended binders with proportion of recycling agent in the blends, it can be easily used in selection of recycling agent for recycling projects.

A linear [Eq. 4.3] and a quadratic [Eq. 4.4] regression model were considered for studying the changes in the complex shear modulus and phase angle with changes in proportion of recycling agent. The general forms of these models can be written as follows:

$$Y = \beta_0 + \beta_1 X + \varepsilon \qquad [4.3]$$

$$Y = \beta_0 + \beta_1 X + \beta_2 X^2 + \varepsilon \qquad [4.4]$$

where:

Y is the response variable (complex shear modulus, phase angle, stiffness, and m-value)

X is the proportion (percentage by mass) of the recycling agent in binder.

The parameters (β_0, β_1 and β_2) were determined by applying the Least Square Method (LSM) to the test results. In LSM method RSS, the sum of the square of the differences between the observations (Y_i) and the predictions ($\hat{Y}_i$), is minimized

$$RSS = \sum_{i=1}^{n}(Y_i - \hat{Y}_i)^2 \qquad [4.5]$$

The linear and non-linear models were compared and checked for goodness of fit. An indicator of how well an estimated regression fits the observed Y_i is the *coefficient of determination* (R^2), which is calculated with the following formula:

$$R^2 = 1 - \frac{RSS}{\sum_{i=1}^{n}\left(Y_i - \bar{Y}_i\right)^2} \qquad [4.6]$$

where:

$$\bar{Y} = \frac{\sum_{i=1}^{n} Y_i}{n}$$

The R^2 should be used with caution, since it is possible to obtain larger R^2 by adding enough terms to the model. The magnitude of the R^2 also depends on the range of the variability in the regression variables. To compare the linear and the quadratic or cubic models, it is always expected that the linear models have the smaller R^2 value. The reason for this is that the non-linear model, such as a quadratic model, has additional parameters. The more estimated parameters, the better the model will fit the sample data. However, the models with more parameters are more complicated and less applicable.

Another indicator for evaluating the goodness of fit between two models is the *mean standard error of the regression* (*MSE*). The *MSE* is the average standard deviation of the regression line. The larger the MSE for a specific regression model, the poorer the associated predictions. The *MSE* can be calculated from the following formula:

$$MSE = \frac{RSS}{d_{frd}} = \left(\frac{\sum (Y_i - \hat{Y}_i)^2}{n-p} \right)^{1/2} \qquad [4.7]$$

where:

d_{frd} is the degrees of freedom

n is the number of observations

p is number of parameters in model.

The F-test is the best method for comparing two models, null hypothesis (*NH*) to alternative hypothesis (*AH*), such as Equations in [4.3] and [4.4]. The *F-test* is evidence against a smaller model if calculated F from Equation [4.8] is large when compared to the percentage point of $F(d.f._{NH}\text{-}d.f._{AH},\ d.f._{AH})$ (Weisberg, 1980). F is the only criterion that has a significance level associated with it.

$$F = \frac{(RSS_{NH} - RSS_{AH}) / (d.f._{NH} - d.f._{AH})}{RSS_{NH} / d.f._{AH}} \qquad [4.8]$$

The R^2, *MSE*, and F-test were used to evaluate the goodness of fit of the linear and non-linear models for blended binders.

4.2.1 Models for Shear Modulus and Stiffness of Blended Binders

The first step for modeling blended binders was measuring the complex shear modulus and phase angle of virgin and aged blended binders with the Dynamic Shear Rheometer at moderate and high pavement temperatures (7 to 70°C). The stiffness and m-value of the aged binders were measured with Bending Beam Rheometer at low pavement temperatures (-18 to –30°C). The test results and discussion related to precision of these measurements were presented in Chapter Three. The best straight lines and the best quadratic curves (at different temperatures and aging conditions) were found for complex shear modulus and stiffness of all blended binders by LSM and the values of R^2 and *MSE* were compared. The *F-test* was conducted to determine whether models were significantly different from each other.

The lowest coefficient of determination for shear modulus regression models was related to the linear model of the unaged binder blended with Flexon equal to 0.79. All other coefficients of determination, for linear and non-linear models, were higher than 0.939. The minimum R^2 of stiffness was attributed to linear model of binder blended with Flexon equal to 0.91 (at –24°C). This means that both linear and non-linear models had high coefficients of determination. The R^2s were higher for binders blended with asphalt cements (200-300 and 300-400) than binders blended with Cyclogen and Flexon. This could be attributed to more uniformity of binders blended with asphalt cements compared to the recycling agents.

In 18 of 56 different cases of models for complex shear modulus, the *MSE* of the linear models were less than that of the non-linear models. In six of 12 different models (three temperatures and four recycling agents) for stiffness, the *MSE* of the linear models were less than the *MSE* of non-linear models. The average *MSE* of aged shear modulus was less than average *MSE* of unaged shear modulus for both linear and non-linear models.

Both R^2 and *MSE* indicate that a linear model can predict the complex shear modulus and stiffness of the blended binders with change in the proportion of recycling agent. The coefficients of determination and standard errors of regression for the linear and non-linear models are presented in Table 4.1. The values presented in Table 4.1 are the average values of R^2 and *MSE* for all tested temperatures (11 temperatures for virgin G^*, three temperatures for aged G^*, and three temperatures for S), two aging conditions (aged and virgin) and four recycling agent materials used in this study.

The*F-test* was used for comparing the linear model (null hypothesis) to a non-linear model (alternative hypothesis). The *F-values* were calculated, based on Eq. [4.8], for all cases. The *F-test* indicates that the non-linear models did not show any superiority over the linear model for prediction of complex shear modulus and stiffness of blended binders.

The above statistical analysis showed that a linear relationship is accurate enough for predicting the complex shear modulus and stiffness of blended binders (for all pavement temperatures and aging conditions) with percentage by mass of recycling agent in blends. This linear relationship for complex shear modulus can be presented as:

$$log\ G^*_{Blended} = log\ G^*_{Aged} + X.\left(log\ G^*_{Recycling\ Agent} - log\ G^*_{Aged}\right) \qquad [4.9]$$

where X is the proportion of recycling agent (percentage by mass) in the blend. This model can be further simplified into:

$$G^*_{Blend} = \left[G^*_{Re cycling\ agent}\right]^{x)} . \left[G^*_{Aged\ asphalt}\right]^{1-x)} \qquad [4.10]$$

Similarly, the same model can be written for the stiffness of blended binders as follows:

Table 4.1 Summary of Regression Analysis for Complex Shear Modulus and Stiffness of Blended Binders

PG Parameters	G* (Unaged)			G* (Aged)			S		
Model Evaluation Criteria	R^2	MSE (log Pa)	F*	R^2	MSE (log Pa)	F*	R^2	MSE (log MPa)	F*
Linear Model	0.96	0.0994	64.9	0.9797	0.0470	90.9	0.96	0.0505	4.3
Non-Linear Model	0.97	0.0635		0.9922	0.0367		0.99	0.0161	

F (0.05; 1, 1) = 161.4

F* are the average of calculated F values for all temperatures and recycling agents material

$$\log S_{Blended} = \log S_{Aged} + X \cdot \left(\log S_{Recycling\,Agen} - \log S_{Aged}\right) \qquad [4.11]$$

or

$$S_{Blend} = \left[S_{Recycling\ agent}\right]^{X_l} \cdot \left[S_{Aged\ asphalt}\right]^{l-X_l} \qquad [4.12]$$

4.2.2 Temperature Dependency of Blended Lines for G* and S

Figures 4.1 and 4.2 depict the linear relationship for change in unaged and aged complex shear modulus with proportion of recycling agent of binders blended with 200-300 asphalt cement at high and intermediate pavement temperatures. Figures 4.3 to 4.8 show the linear relationship for the same variables for binders blended with 300-400, Cyclogen, and Flexon respectively. Figures 4.9 to 4.12 present the linear relationship for change in stiffness with proportion of recycling agent of binders blended with 200-300, 300-400, Cyclogen and Flexon. These figures suggest that linear regression lines are nearly parallel to each other.

The parallelism of the regression lines or temperature dependency of the complex shear modulus and stiffness lines in Figures 4.1 to 4.12 was studied with the following statistical analysis (Seber, 1977).

K regression lines can be written as follows:

$$Y = \alpha_k + \beta_k x_k + \varepsilon \qquad (k = 1, 2, \ldots\ldots, K) \qquad [4.13]$$

with n_k pair of observations (x_{ki}, Y_{ki} $i=1,2,\ldots\ldots n_k$) on the *k*th line, the model will be:

$$Y_{ki} = a_k + \beta_K x_{Ki} + \varepsilon_{ki} \qquad (i = 1, 2, \ldots\ldots, n_k) \qquad [4.14]$$

The general form of these equations is $Y = X\gamma + \varepsilon$. In a matrix form it could be written:

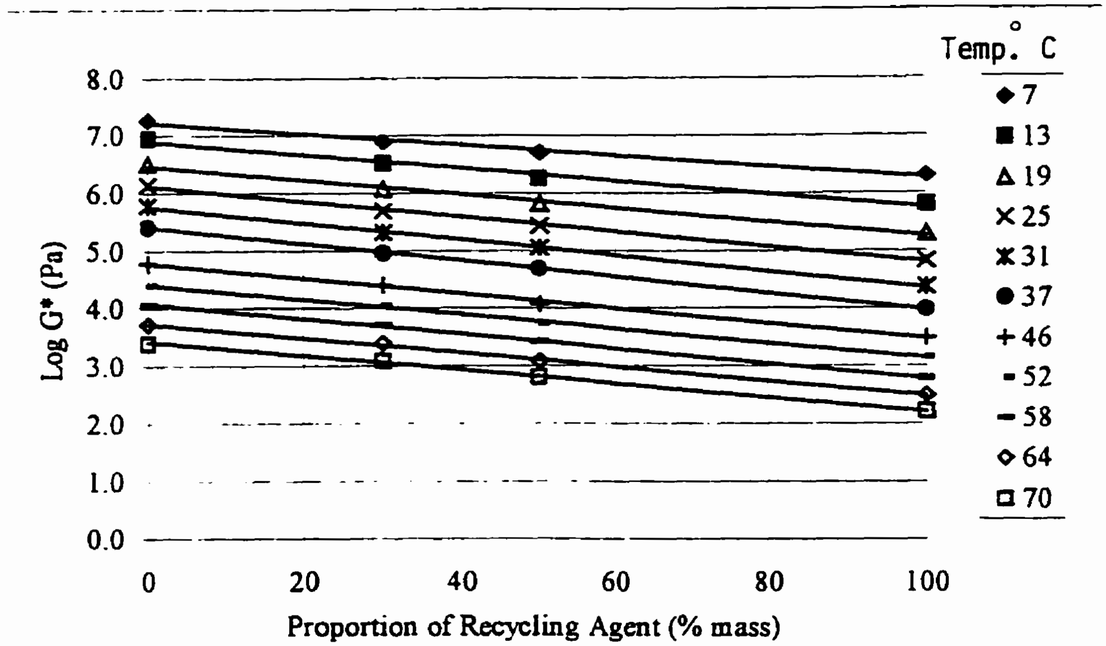


Figure 4.1 Change in Complex Shear Modulus with Proportion of Recycling Agent for Unaged Binders Blended with 200-300 Asphalt Cement at Intermediate and High Pavement Temperatures

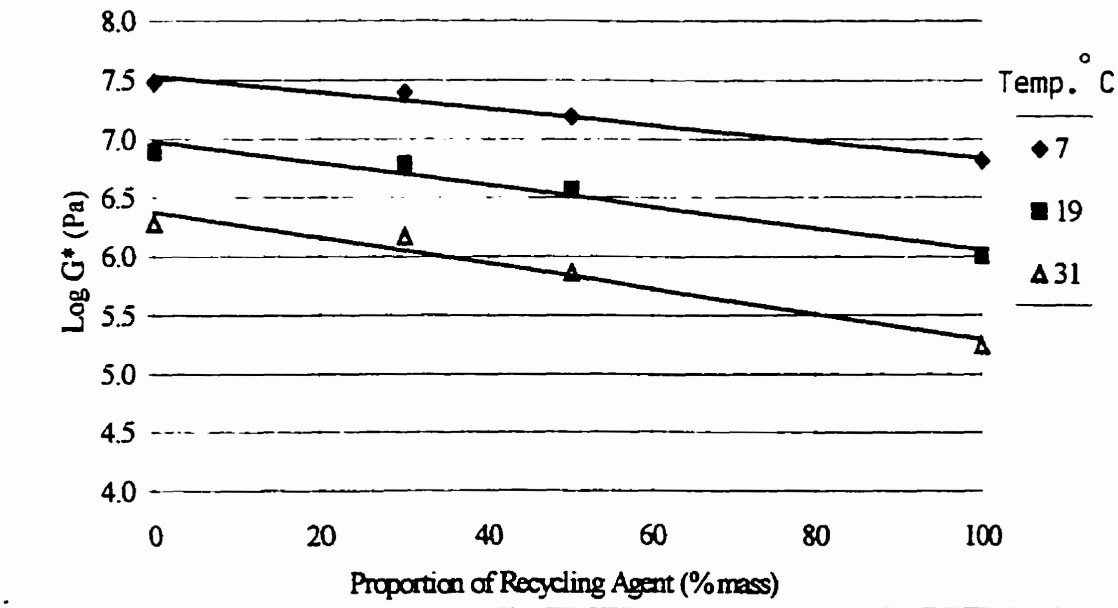


Figure 4.2 Change in Complex Shear Modulus with Proportion of Recycling Agent for Aged Binders Blended with 200-300 Asphalt Cement at Intermediate Pavement Temperatures

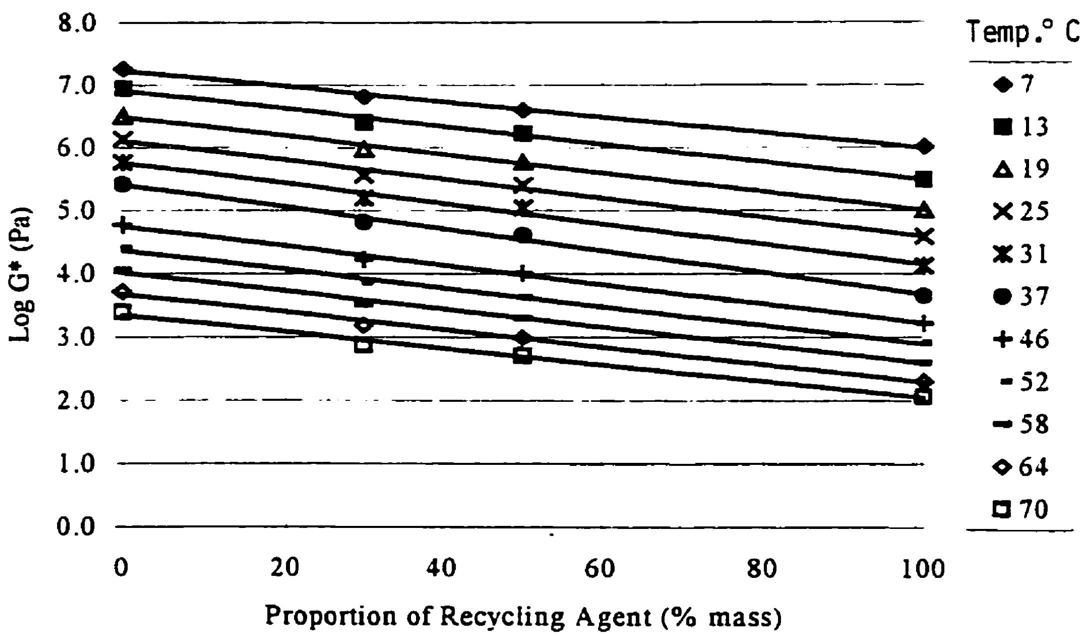


Figure 4.3 Change in Complex Shear Modulus with Proportion of Recycling Agent for Unaged Binders Blended with 300-400 Asphalt Cement at Intermediate and High Pavement Temperatures

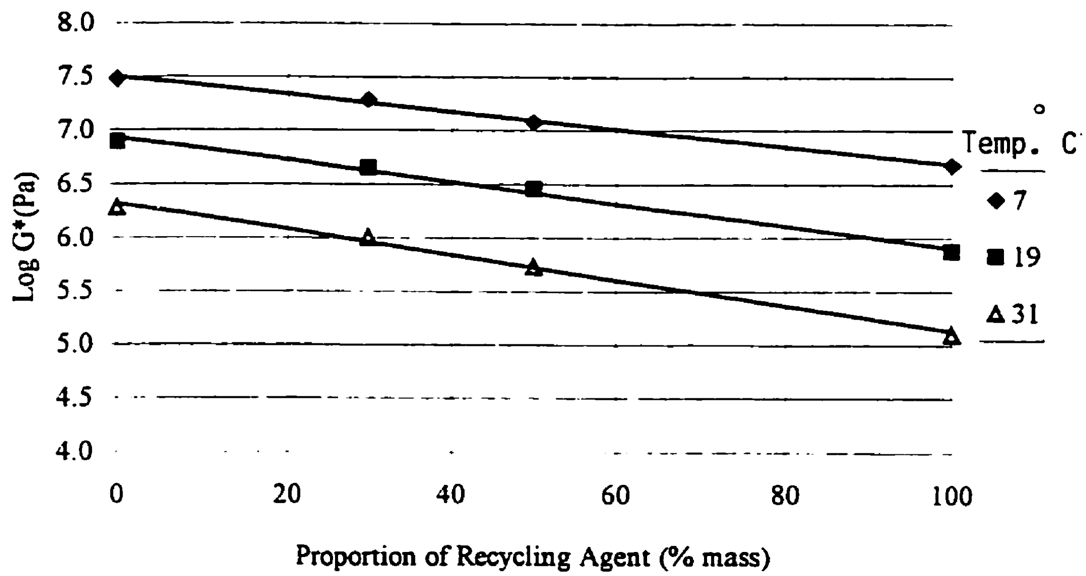


Figure 4.4 Change in Complex Shear Modulus with Proportion of Recycling Agent for Aged Binders Blended with 300-400 Asphalt Cement at Intermediate Pavement Temperatures

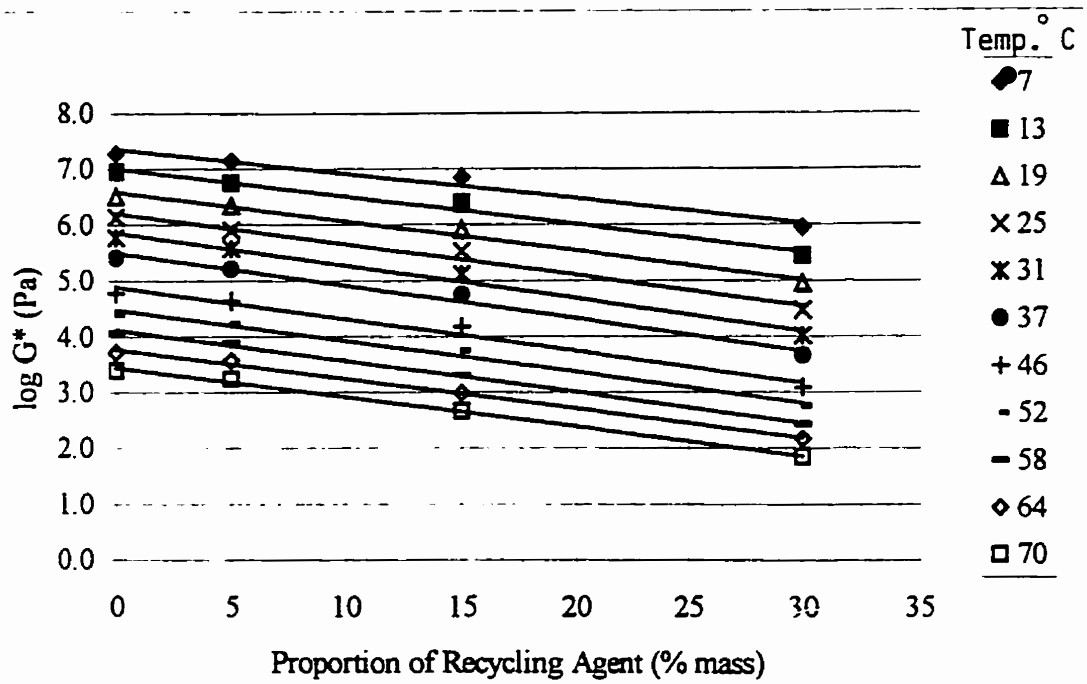


Figure 4.5 Change in Complex Shear Modulus with Proportion of Recycling Agent for Binders Blended with Cyclogen at Intermediate and High Pavement Temperatures

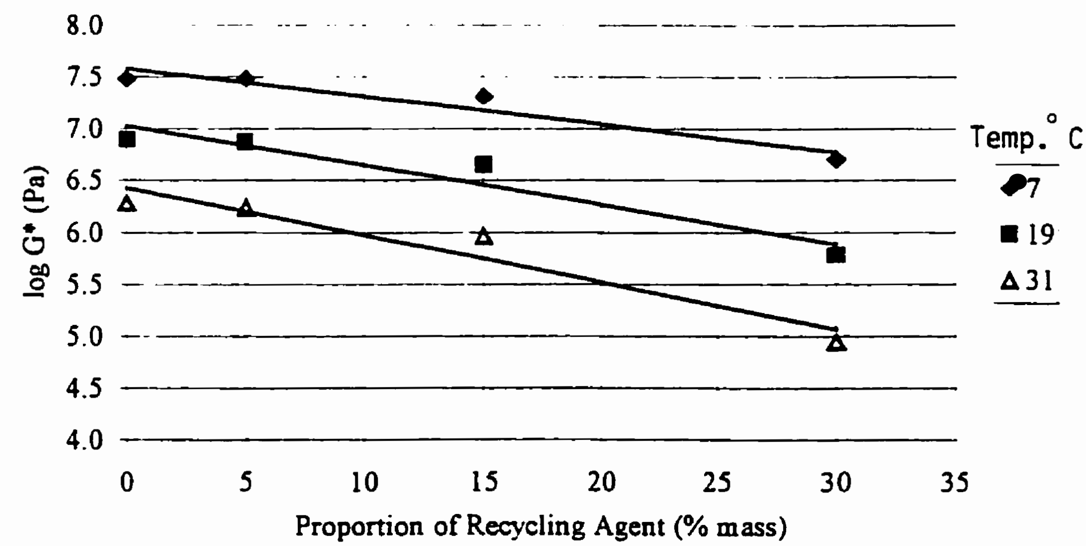


Figure 4.6 Change in Complex Shear Modulus with Proportion of Recycling Agent for Aged Binders Blended With Cyclogen at Intermediate Pavement Temperatures

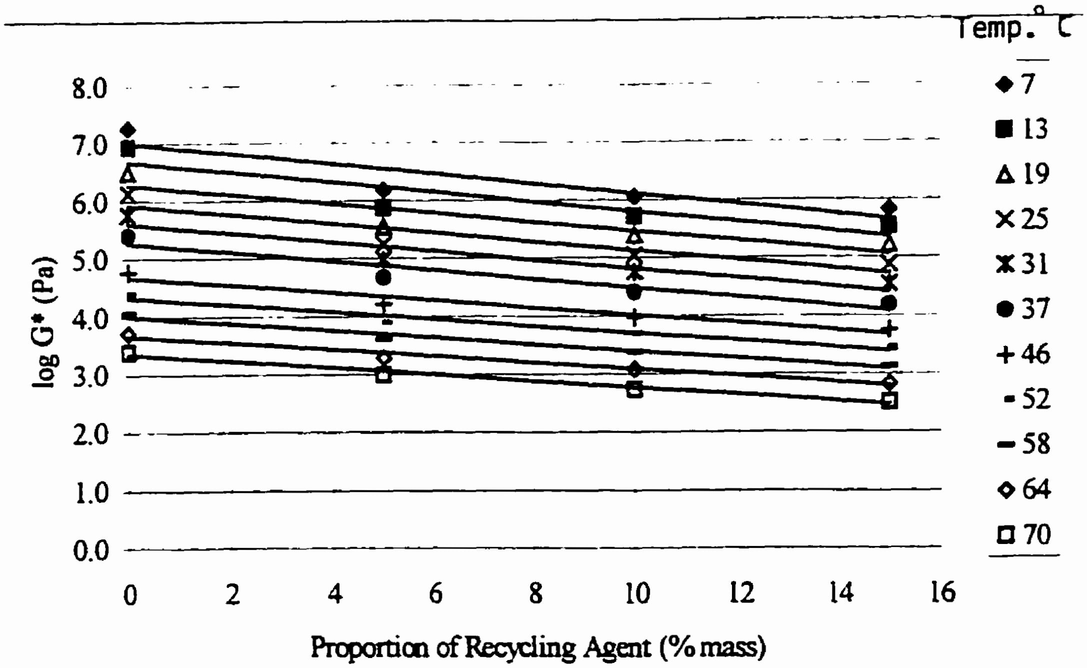


Figure 4.7 Change in Complex Shear Modulus with Proportion of Recycling Agent for Virgin Binders Blended With Flexon at Intermediate and High Pavement Temperatures

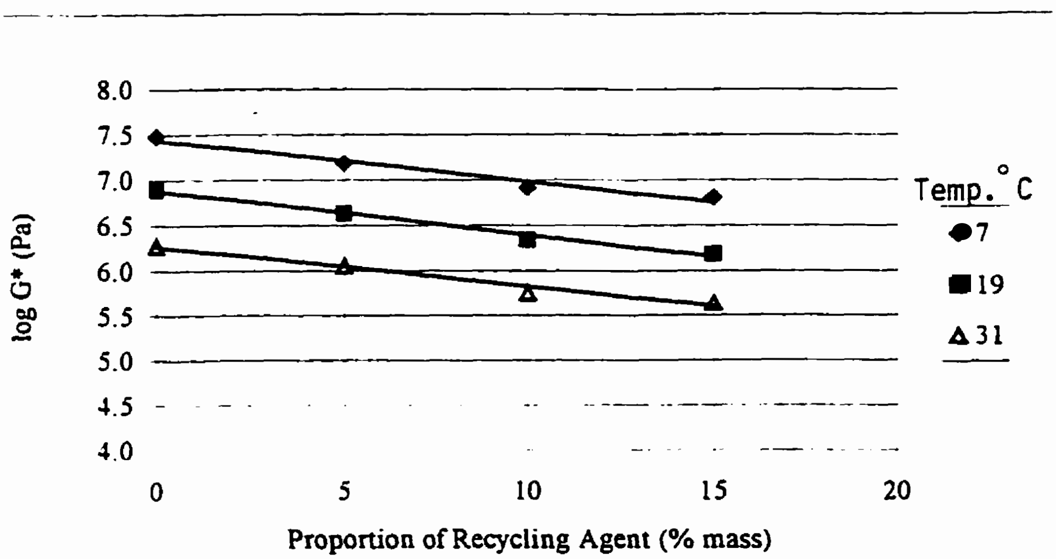


Figure 4.8 Change in Complex Shear Modulus with Proportion of Recycling Agent for Aged Binders Blended With Flexon at Intermediate Pavement Temperatures

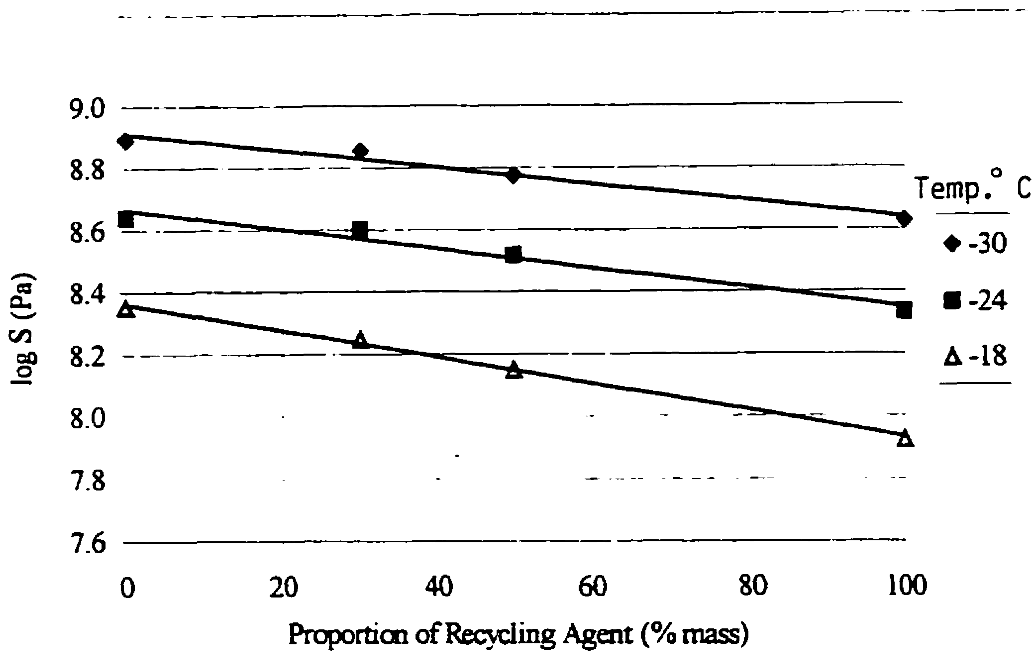


Figure 4.9 Change in Stiffness with Proportion of Recycling Agent for Binders Blended with 200-300Asphalt Cement at Low Pavement Temperatures

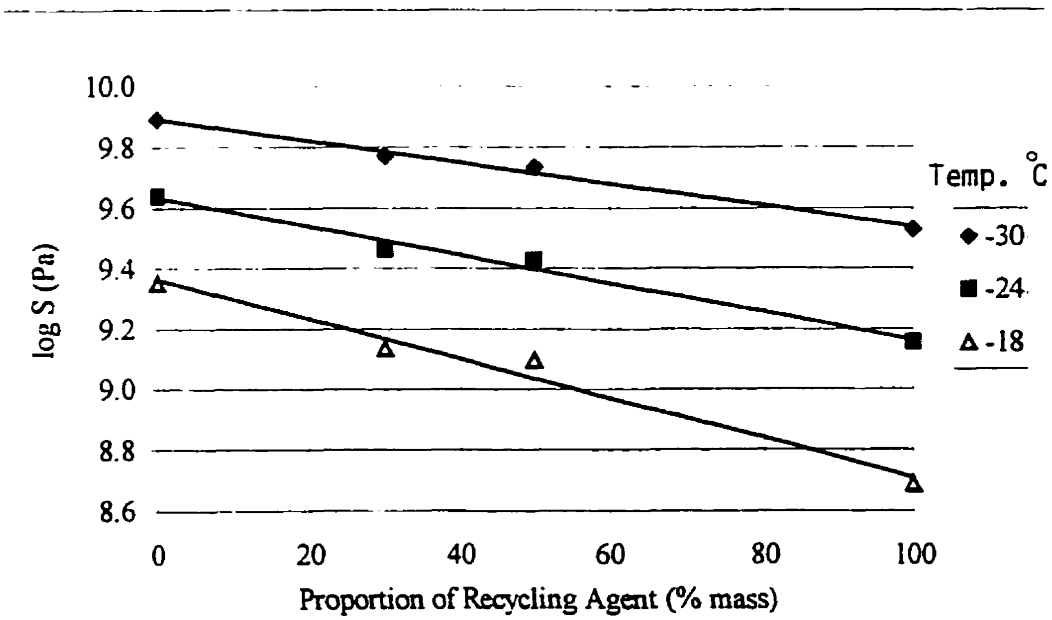


Figure 4.10 Change in Stiffness with Proportion of Recycling Agent for Binders Blended with 300-400Asphalt Cement at Low Pavement Temperatures

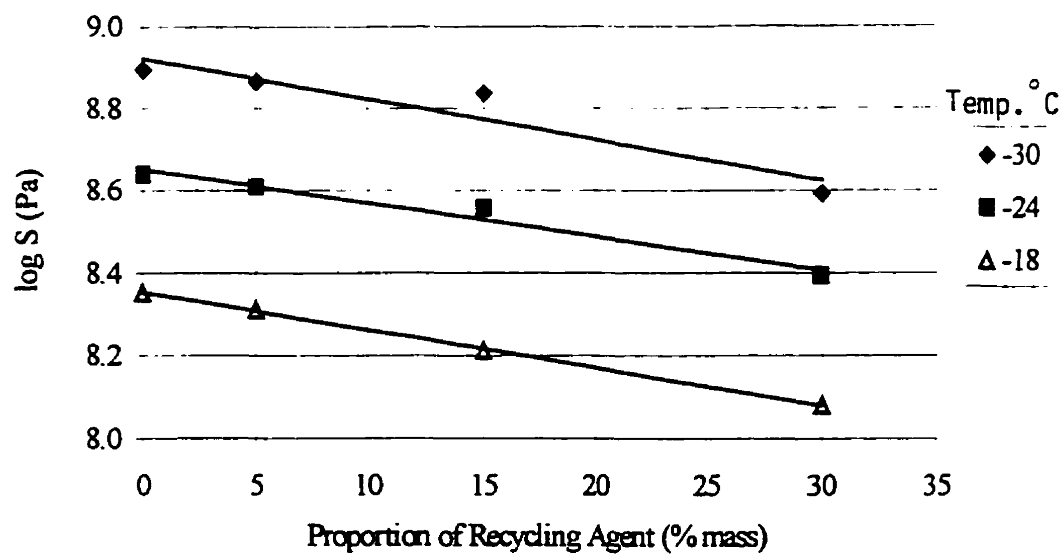


Figure 4. Change in Stiffness with Proportion of Recycling Agent for Binders Blended with Cyclogen at Low Pavement Temperatures

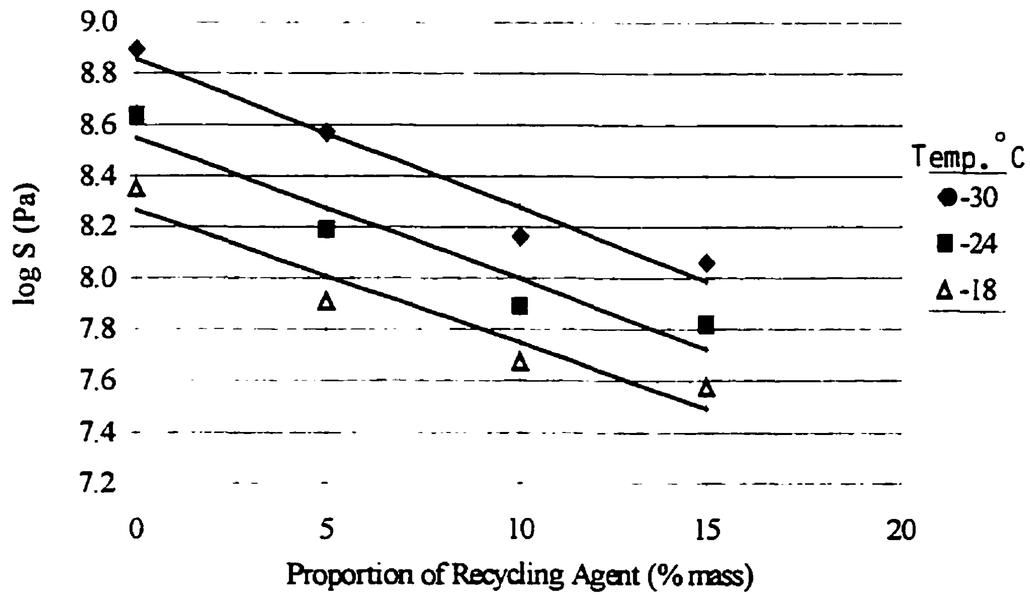


Figure 4.12 Change in Stiffness with Proportion of Recycling Agent for Binders Blended with Flexon at Low Pavement Temperatures

$$
\mathbf{X}\gamma = \begin{bmatrix}
1 & 0 & \cdots & 0 & x_{11} & 0 & \cdots & 0 \\
1 & 0 & \cdots & 0 & x_{12} & 0 & \cdots & 0 \\
\cdots & \cdots & \cdots & \cdots & \cdots & \cdots & \cdots & \cdots \\
1 & 0 & \cdots & 0 & x_{1n_1} & 0 & \cdots & 0 \\
0 & 1 & \cdots & 0 & 0 & x_{21} & \cdots & 0 \\
0 & 1 & \cdots & 0 & 0 & x_{22} & \cdots & 0 \\
\cdots & \cdots & \cdots & \cdots & \cdots & \cdots & \cdots & \cdots \\
0 & 1 & \cdots & 0 & 0 & x_{2n_2} & \cdots & 0 \\
\cdots & \cdots & \cdots & \cdots & \cdots & \cdots & \cdots & \cdots \\
0 & 0 & \cdots & 1 & 0 & 0 & \cdots & x_{k1} \\
0 & 0 & \cdots & 1 & 0 & 0 & \cdots & x_{k1} \\
\cdots & \cdots & \cdots & \cdots & \cdots & \cdots & \cdots & \cdots \\
0 & 0 & \cdots & 1 & 0 & 0 & \cdots & x_{kn_k}
\end{bmatrix}
\begin{bmatrix}
\alpha_1 \\ \alpha_2 \\ \cdots \\ \alpha_K \\ \beta_1 \\ \beta_2 \\ \cdots \\ \beta_K
\end{bmatrix} \tag{4.15}
$$

It is possible to test any hypothesis of the form $H\!: A\gamma = C$ by using the general regression theory where C can be a constant or a function. This study was interested in investigating whether the K lines are parallel, $A\gamma = 0$, as the hypothesis being:

$$
H_1\!: \beta_1 = \beta_2 = \cdots \beta_K \text{ or } \beta_1 - \beta_K = \beta_2 - \beta_K = \cdots = \beta_{K-1} - \beta_K = 0 \tag{4.16}
$$

This hypothesis can also be written in matrix form as follows:

$$
\left[0 \left| \begin{matrix}
1 & 0 & 0 & \cdots & 0 & -1 \\
0 & 1 & 0 & \cdots & 0 & -1 \\
\cdots & \cdots & \cdots & \cdots & \cdots & \cdots \\
0 & 0 & 0 & 0 & 1 & -1
\end{matrix} \right. \right]
\begin{pmatrix} \alpha \\ \beta \end{pmatrix} = 0 \tag{4.17}
$$

The statistical test for H_1 is:

$$
F = \frac{\left(RSS_{H_1} - RSS\right)/(K-1)}{RSS/(N-2K)} \tag{4.18}
$$

In this formula, RSS is simply the sum of the residual sums of squares for each regression, namely,

$$RSS = \sum_{K=1}^{K} \left\{ \sum_{i=1}^{n_k} \left(Y_{Ki} - \overline{Y}_k\right)^2 - \hat{\beta}_K^2 \sum_{i=1}^{n_k} \left(x_{Ki} - \overline{x}_K\right)^2 \right\} \qquad [4.19]$$

where

$$\hat{\beta}_K = \frac{\sum_i \left(Y_{Ki} - \overline{Y}_K\right)\left(x_{Ki} - \overline{x}_K\right)}{\sum_{i=1} \left(x_{Ki} - \overline{x}_K\right)^2} \qquad [4.20]$$

$$RSS_{H_1} - RSS = \sum_{K=1} \hat{\beta}_K \sum_i \left(x_{Ki} - \overline{x}_K\right)^2 - \tilde{\beta}^2 \sum_K \sum_i \left(x_{Ki} - \overline{x}_K\right)^2 \qquad [4.21]$$

$$\tilde{\beta} = \frac{\sum_K \sum_i \left(Y_{Ki} - \overline{Y}_K\right)\left(x_{Ki} - \overline{x}_K\right)}{\sum_K \sum_i \left(x_{Ki} - \overline{x}_K\right)^2} \qquad [4.22]$$

F_{crit} is obtained for K-1 and N-2K degree of freedom from *F-test* table.

Table 4.2 summarizes the results of analysis of parallelism of regression lines for complex shear modulus and stiffness for all blended binders ($\alpha = 0.05$).

Table 4.2- Summary of Statistical Analysis of Parallelism of the Regression Lines for Complex Shear Modulus and Stiffness

Parameter	**Recycling Agent**	**Aging Conditions**	***K***	***N***	***F***	$F_{crit.}$
G^*	200-300	Unaged	11	44	5.02	2.75
G^*	300-400	Unaged	11	44	2.58	2.75
G^*	Cyclogen	Unaged	11	44	0.81	2.75
G^*	Flexon	Unaged	11	44	0.36	2.75
G^*	200-300	Aged	3	12	2.30	19.33
G^*	300-400	Aged	3	12	11.9	19.33
G^*	Cyclogen	Aged	3	12	1.51	19.33
G^*	Flexon	Aged	3	12	0.17	19.33
S	200-300	Aged	3	12	8.98	19.33
S	300-400	Aged	3	12	6.39	19.33
S	Cyclogen	Aged	3	12	0.34	19.33
S	Flexon	Aged	3	12	0.10	19.33

G^* (Unaged) $F(0.05 ; 10, 22) = 2.75$

G^* (Aged) $F(0.05 ; 2, 6) = 19.33$

S $F(0.05 ; 2, 6) = 19.33$

K = Number of temperatures or lines

N = Total number of observations

The above analysis shows that in all studied cases (PG parameters, recycling agents and aging conditions) except one case (unaged G^* for 200-300) the $F_{crit} > F$. It could be concluded that the regression lines are parallel to each other. Shift value, which is the average of vertical distance between regression lines in Figures 4.1 to 4.12 can be used for prediction of complex shear modulus and stiffness at any temperature other than the tested temperatures and for any proportion of recycling agent in blends. This allows characterizing of blended binders at different temperatures. The shift values were calculated and the results are presented in Table 4.3. The shift values reported in Table 4.3 are dependent on aging conditions, testing methods, and different range of testing temperatures therefore, it is not possible to compare these values to each other. For all cases, binders blended with Flexon had the lowest and binder blended with 300-400 asphalt cement had the highest values of shift values. The standard deviation of shift values, in Table 4.3, indicate that the shift values could be used with enough accuracy for predicting the complex shear modulus or stiffness at temperatures other than the testing temperatures.

4.2.3 Models for Phase Angle and m-value of Blended Binders

The other parameters that are used in the PG system are the phase angle and m-value. The phase angle is an indicator of the relative amount of elastic and plastic deformation. The m-value is the slope of the log of stiffness versus loading times at 60-seconds.

The same statistical analysis for complex shear modulus was repeated for phase angle results. Similar to the complex shear modulus, the lowest coefficient of determination was related to the unaged binders blended with Flexon. All other cases had a R^2 value more than 0.91. In 25 cases from 56 different cases of the aging conditions and temperatures, the *MSE* values of the linear models of phase angles were less than the non-linear models. In other cases the *MSE* of the non-linear regression models were lower than the linear models. Table 4.4 presents the average values of the R^2 and *MSE* of the phase angle and m-value for all blended binders.

Table 4.3- Shift Values* for Binders Blended with Recycling Agents (RA)

Recycling Agent	Statistical Parameters	G* (Unaged) log Pa	G* (Aged) log Pa	S log MPa
200-300	Mean	0.35205	0.33280	0.3117
	S. D.	0.035192	0.004137	0.063498
300-400	Mean	0.372967	0.38814	0.3384
	S. D.	0.054669	0.009157	0.033234
Cyclogen	Mean	0.382622	0.34732	0.2776
	S. D.	0.050847	0.0030706	0.049285
Flexon	Mean	0.325528	0.29095	0.2712
	S. D.	0.014374	0.000813	0.03182

* Shift Value is the average of vertical distance between regression lines of G* and S, in Figures 4.1 to 4.12, for 6°C change in temperature.

S. D.= Standard Deviation

N = Number of calculated shift values between two temperatures (N=11 for G* Unaged, N=2 for G* Aged, and N=2 for S)

Both linear and non-linear models can predict the phase angle and *m-value* with reasonable accuracy. The *MSE* values for both models were less than 1.32 degree for the phase angle and less than 0.0085 for the m-value. Although the non-linear model has little higher accuracy, in the interests of simplicity, the linear model could be accepted with a reasonable accuracy. The *F-test* strongly suggests that the quadratic model has no superiority to the linear model for determining both phase angles and m-values of blended binders.

The above statistical analysis showed that a linear line is accurate enough for prediction of phase angle and m-value of blended binders with percentage mass of recycling agent in the blend. Numerically, the phase angle can be expressed as:

$$\delta_{Blended} = \delta_{Aged} + X \,.\left(\delta_{Recycling\ Agen} - \delta_{Aged}\right) \qquad [4.23]$$

and similarly for *m-value* as:

$$m_{Blended} = m_{Aged} + X \,.\left(m_{Recycling\ Agent} - m_{Aged}\right) \qquad [4.24]$$

These models can be rewritten as below:

$$\delta_{Blended} = X.\delta_{Recycling\ Agent} + (1 - X).\delta_{Aged} \qquad [4.25]$$

or

$$m_{Blended} = X.m_{Recyclin\ Agent} + (1 - X).m_{Aged} \qquad [4.26]$$

where X is the proportion of recycling agent (percentage by mass) in the blend.

Table 4.4 Summary of Regression Analysis of Linear and Non-Linear Models for Phase Angle and m-Value of Blended Binders

PG Binder Parameters	δ (Unaged)			δ (Aged)			m-value		
Model Evaluation Criteria	R^2	MSE (Degree)	F*	R^2	MSE (Degree)	F*	R^2	MSE	F*
Linear Model	0.96	1.1583	30.36	0.95	1.9117	155.56	0.96	0.0084	139.56
Non-Linear Model	0.98	1.1363		0.99	0.5845		0.99	0.0035	

F (0.05; 1, 1) = 161.4

F* are the average of calculated F values for all temperatures and recycling agent material

4.2.4 Temperature Dependency of Blended Lines for δ and m-value

Figures 4.13 to 4.24 show the linear relationship for change in the phase angle and m-value versus proportion of recycling agents for all binders used in this study. Unlike the complex shear modulus, the lines are not parallel to each other. The phase angles inclined to 90° at high temperatures and with a high proportion of the recycling agent. The phase angle and m-value of blended lines, in Figures 4.13 to 4.24, are temperature-dependent. A temperature dependency study could help to predict the phase angle and m-value of the blended binders at other temperatures than tested temperatures. The temperature dependencies of the phase angle and m-value of blended binders were studied by finding the relationship of phase angles with temperatures. Therefore two models, a logarithmic equation [4.27] and a power model equation [4.28], were compared for studying temperature dependency of the phase angle and m-value of the blended lines. Equations [4.27] and [4.28] are similar to each other but there is a log X transformation in the power model compared to the logarithmic model.

$$X = \alpha_1 + \beta_1 \ln T \qquad [4.27]$$

$$X = \alpha_2 . T^{\beta_2} \qquad [4.28]$$

Where:

T is the temperature (°K)

X is the dependent variable, which is the phase angle or m-value of blended binders.

The parameters of models were estimated with test results for all blended binders. The R^2 values were calculated. Table 4.5 depicts the average values of R^2 for logarithmic and power models of phase angle and m-value. The average R^2 values of the power and logarithmic models were very close to each other for phase angle of unaged and aged blended binders. The lowest coefficients of determination were related to F3 binder blended (15 percent of weight Flexon) equal to 0.85.

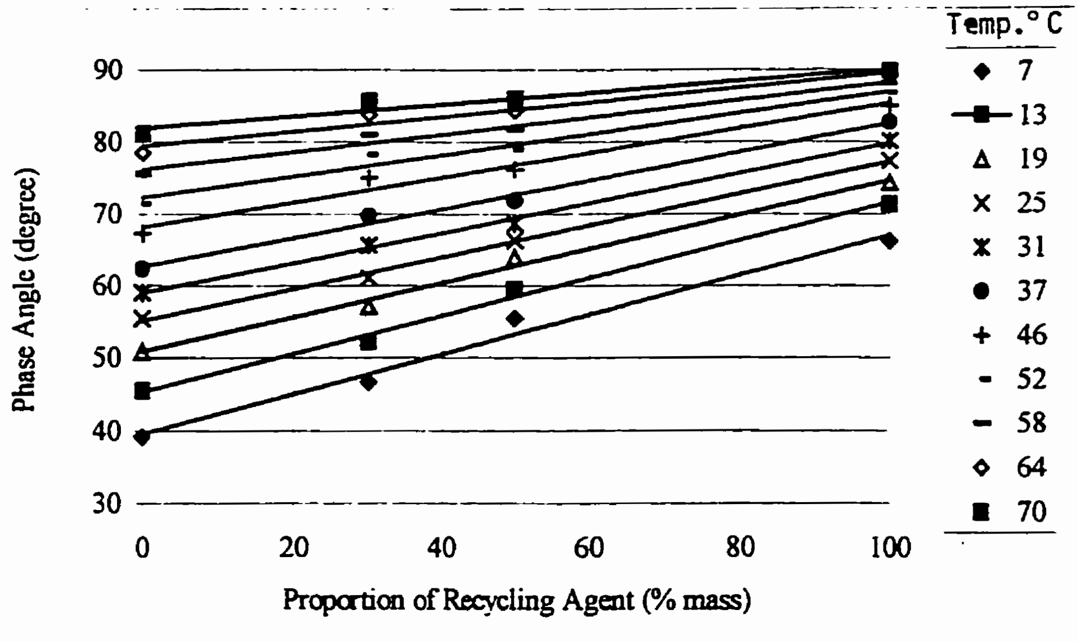


Figure 4.13 Change in Phase Angle with Proportion of Recycling Agent for Unaged Binders Blended with 200-300 Asphalt Cement at High and Intermediate Pavement Temperatures

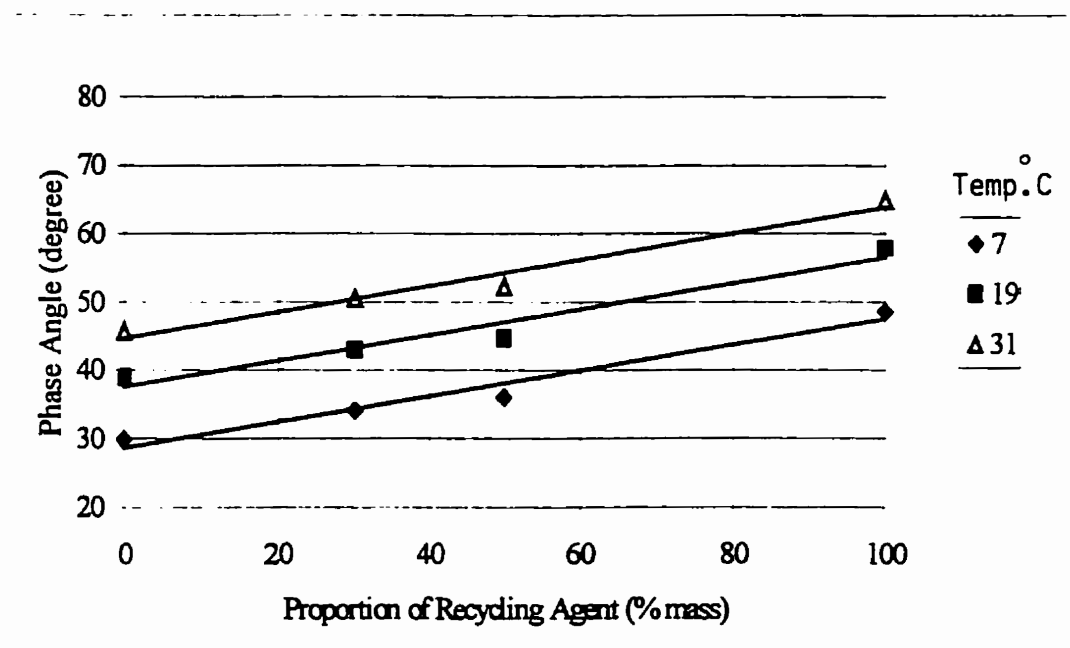


Figure 4.14 Change in Phase Angle with Proportion of Recycling Agent for Aged Binders Blended with 200-300 Asphalt Cement at Intermediate Pavement Temperatures

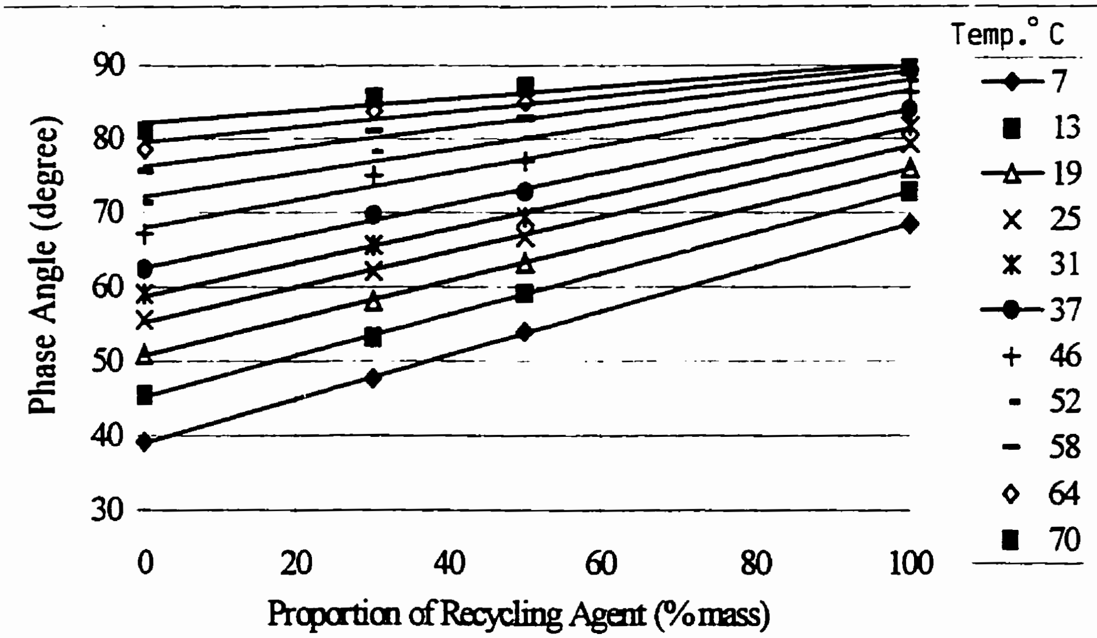


Figure 4.15 Change in Phase Angle with Proportion of Recycling Agent for Unaged Binders Blended with 300-400 Asphalt Cement at High and Intermediate Pavement Temperatures

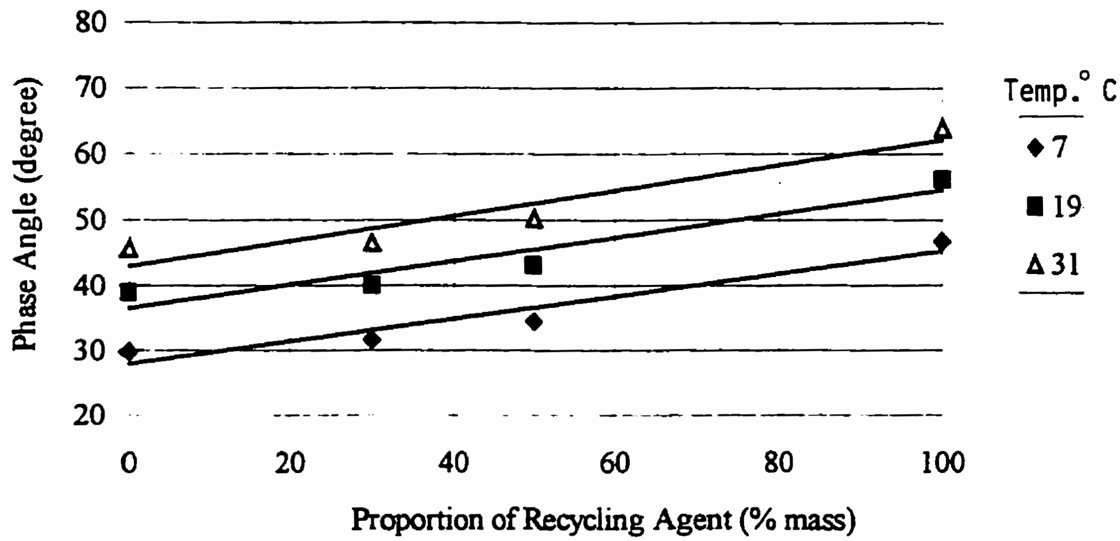


Figure 4.16 Change in Phase Angle with Proportion of Recycling Agent for Aged Binders Blended with 300-400 Asphalt Cement at Intermediate Pavement Temperatures

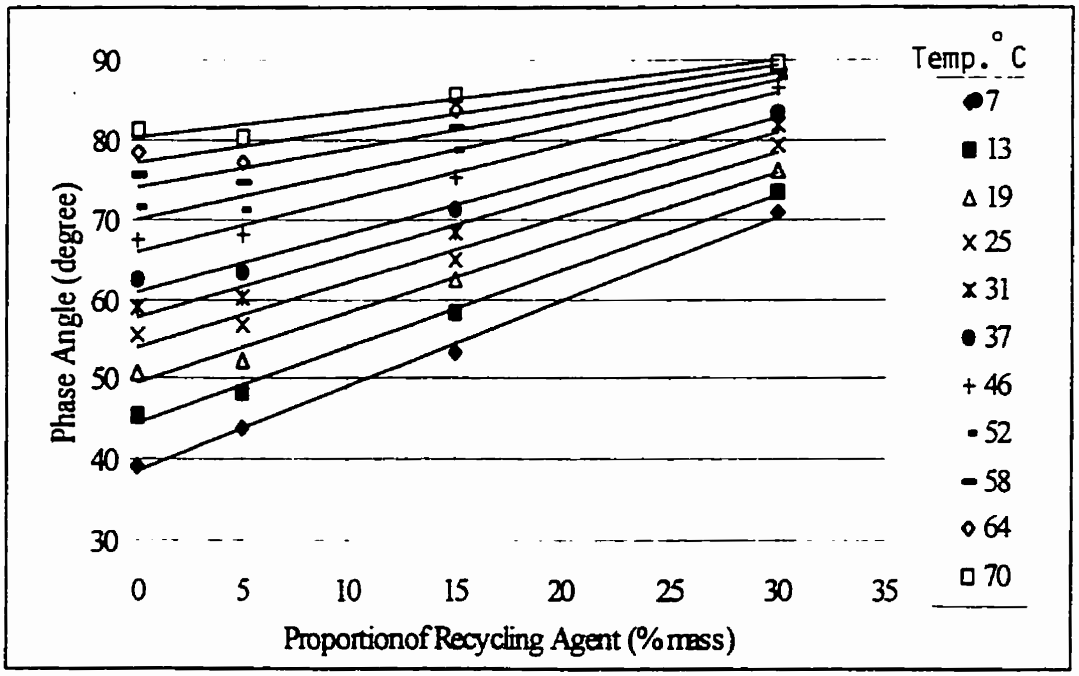


Figure 4.17 Change in Phase Angle with Proportion of Recycling Agent for Unaged Binders Blended with Cyclogen at High and Intermediate Pavement Temperatures

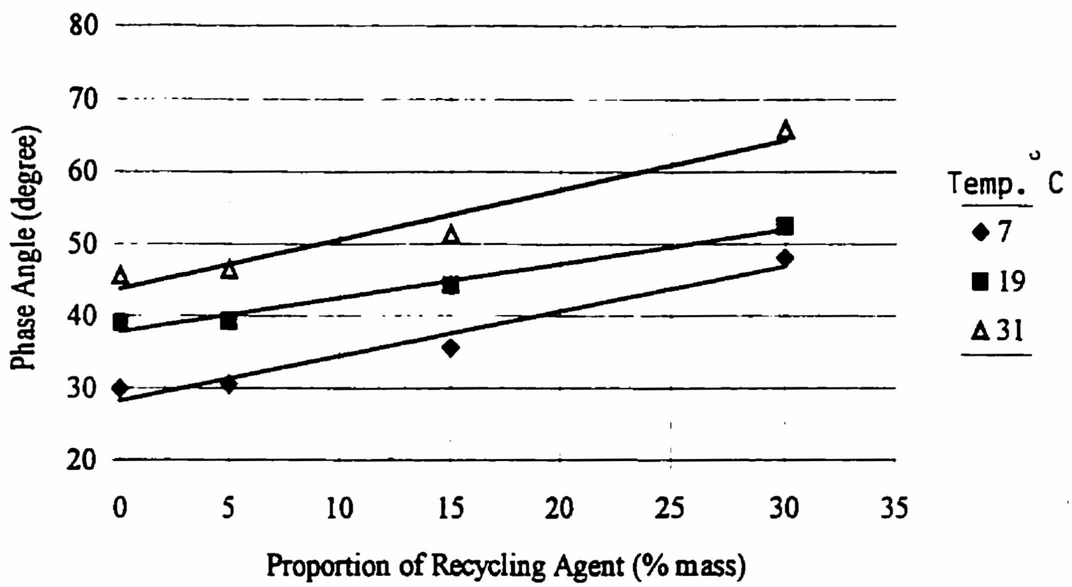


Figure 4.18 Change in Phase Angle with Proportion of Recycling Agent for Aged Binders Blended with Cyclogen at Intermediate Pavement Temperatures

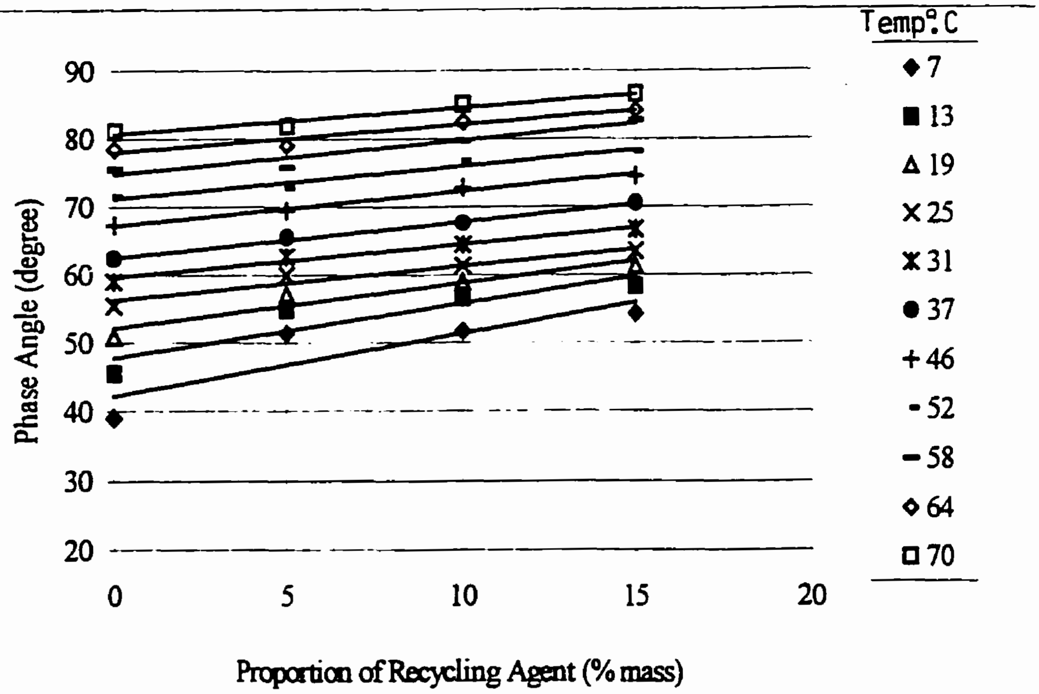


Figure 4.19 Change in Phase Angle with Proportion of Recycling for Unaged Binders Blended with Flexon at High and Intermediate Pavement Temperatures

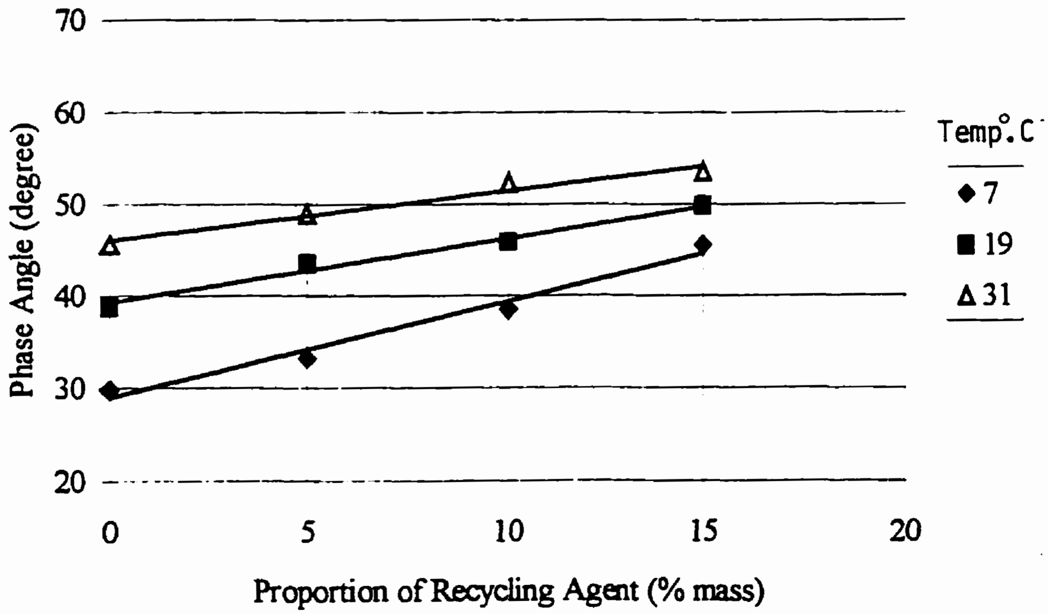


Figure 4.20 Change in Phase Angle with Proportion of Recycling Agent for Aged Binders Blended with Flexon at Intermediate Pavement Temperatures

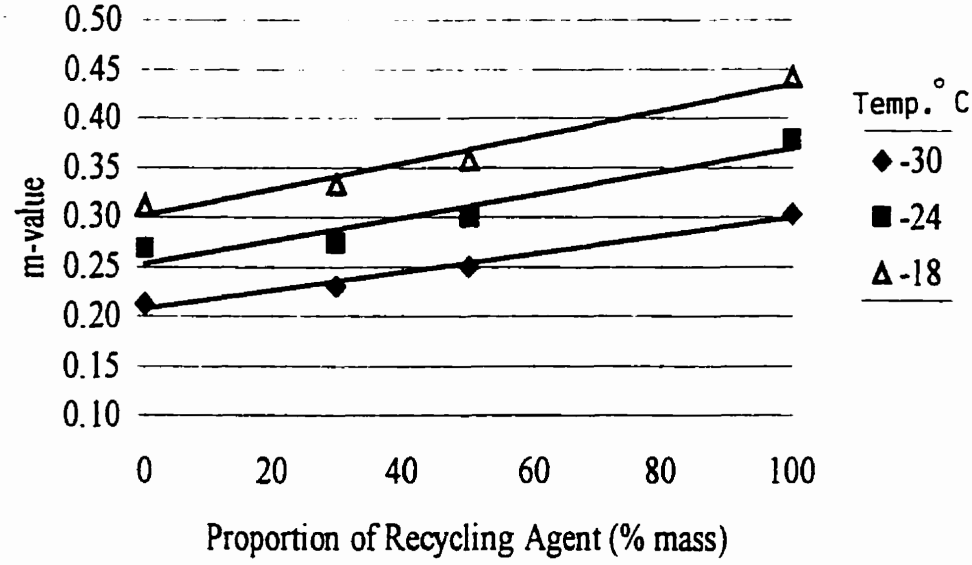


Figure 4.21 Change in m-value with Proportion of Recycling Agent for Binders Blended with 200-300 Asphalt Cement at Low Pavement Temperatures

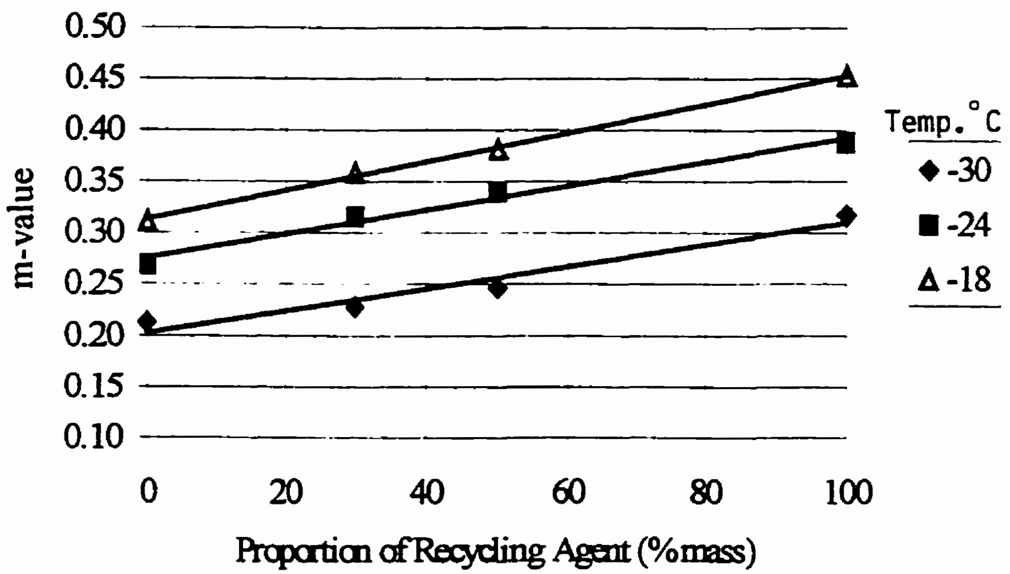


Figure 4.22 Change in m-value with Proportion of Recycling Agent for Binders Blended with 300-400 Asphalt Cement at Low Pavement Temperatures

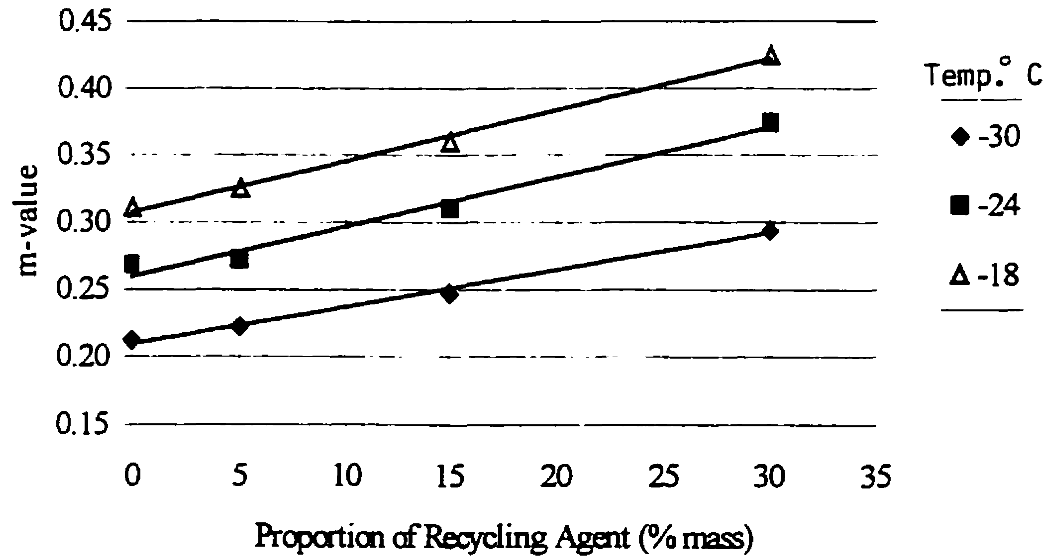


Figure 4.23 Change in m-value with Proportion of Recycling Agent for Binders Blended with Cyclogen at Low Pavement Temperatures

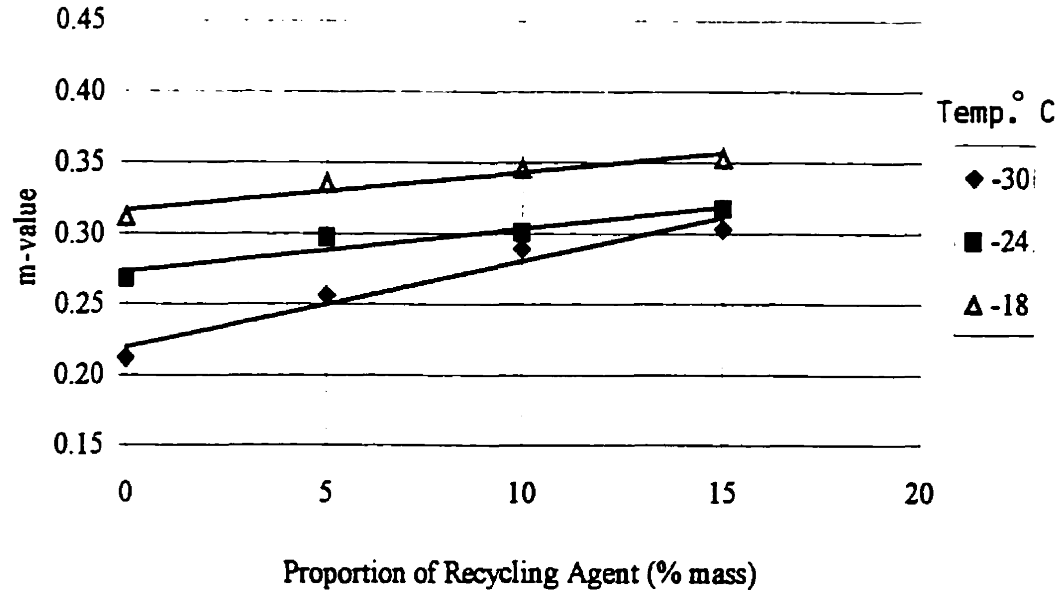


Figure 4.24 Change in m-value with Proportion of Recycling Agent for Binders Blended with Flexon at Low Pavement Temperatures

This analysis suggests that both models, logarithmic and power, can predict the phase angle and m-value with a high correlation. The logarithmic model fits the data with a higher coefficient of determination than the power model. Tables 4.6, 4.7, and 4.8 present the estimated parameters of logarithmic model for phase angle and m-value of unaged and aged blended binders. The estimated parameters were different for various blended binders and a general model with specific parameters can not be suggested for all blended binders.

Table 4.5 Summary of Statistical Analysis for Temperature Dependency of the Phase Angle (Unaged and Aged) and m-value

Models	R^2 for δ (Unaged) N = 44	R^2 for δ (Aged) N = 12	R^2 for m-Value N = 12
Logarithmic	0.98	0.99	0.93
Power	0.97	0.98	0.93

Table 4.6 Estimated Parameters for Unaged Phase Angle Model ($\delta=\alpha+\beta \ln T$) of Blended Binders, Temperature (°K)

Blended Binders	Estimated Parameters	Standard Error	t-Value	R^2	MSE
N1	$\alpha = -1039$	34.660	-29.99	0.99	1.3067
	$\beta = 193.12$	6.038	31.98		
N2	$\alpha = -783.625$	16.855	-46.49	0.99	0.6354
	$\beta = 149.095$	2.936	50.77		
M1	$\alpha = -1004.46$	31.070	-32.32	0.99	1.1714
	$\beta = 187.048$	5.4133	34.55		
M2	$\alpha = -846.29$	25.20	-33.57	0.99	0.95023
	$\beta = 160.082$	4.39127	36.455		
C1	$\alpha = -950.52$	17.0	-55.90	0.99	0.64103
	$\beta = 176.641$	2.962	59.62		
C2	$\alpha = -831.137$	21.13	-39.32	0.99	0.7967
	$\beta = 157.215$	3.681	42.69		
C3	$\alpha = -471.53$	36.33	-12.96	0.96	1.37158
	$\beta = 96.527$	6.338	15.23		
F1	$\alpha = -787.831$	15.857	-49.68	0.99	0.5978
	$\beta = 2148.831$	2.762	53.87		
F2	$\alpha = -950.789$	11.143	-85.32	0.99	0.4201
	$\beta = 177.529$	1.9415	91.44		
F3	$\alpha = -888.82$	29.299	-30.33	0.99	1.1046
	$\beta = 166.237$	5.104	32.566		

MSE is the Mean of Standard errors of Regression Lines

Table 4.7 Estimated Parameters for Aged Phase Angle Model ($\delta=\alpha+\beta \ln T$) of Blended Binders, Temperature (°K)

Blended Binders	Estimated Parameters	Standard Error	t-Value	R^2	MSE
N1	$\alpha=-951.362$ $\beta=174.542$	11.792 2.054	-80.675 84.953	0.99	0.4446
N2	$\alpha=-962.918$ $\beta=177.092$	23.614 4.1142	-40.044 43.044	0.99	0.8904
M1	$\alpha=-1084.73$ $\beta=198.54$	17.346 3.022	-62.53 65.69	0.99	0.6541
M2	$\alpha=-1025.82$ $\beta=188.4874$	15.967 2.781	-64.24 67.75	0.99	0.6020
C1	$\alpha=-931.1178$ $\beta=170.841$	16.131 2.810	-57.72 60.78	0.99	0.6082
C2	$\alpha=-995.681$ $\beta=183.076$	19.07 3.322	-52.20 55.09	0.99	0.7191
C3	$\alpha=-813.38$ $\beta=154.71$	24.28 4.230	-33.49 36.57	0.99	0.9155
F1	$\alpha=-875.908$ $\beta=160.715$	19.51 3.399	-44.89 47.28	0.99	0.7356
F2	$\alpha=-958.766$ $\beta=175.58$	35.079 6.111	-27.33 28.73	0.98	1.3226
F3	$\alpha=-767.794$ $\beta=140.8089$	33.308 5.803	-23.051 24.264	0.98	1.2558

MSE is the Mean of Standard Errors of Regression Lines

Table 4.8 Estimated Parameters for m Value ($m = \alpha + \beta \ln T$) of Blended Binders, Temperature (°K)

Blended Binders	Estimated Parameters	Standard Error	t-Value	R^2	MSE
N1	$\alpha = -8.892$ $\beta = 1.661$	1.355 0.2447	-6.562 6.785	0.95	0.01701
N2	$\alpha = -7.3483$ $\beta = 1.3856$	1.8977 0.3428	-3.872 4.042	0.89	0.02383
M1	$\alpha = -9.3965$ $\beta = 1.7564$	2.344 0.4235	-4.007 4.146	0.89	0.02944
M2	$\alpha = -9.870$ $\beta = 1.8426$	0.8478 0.1531	-11.643 12.031	0.98	0.01064
C1	$\alpha = -10.265$ $\beta = 1.9095$	0.5215 0.09420	-19.685 20.27	0.99	0.0065
C2	$\alpha = -11.4385$ $\beta = 2.1281$	0.5693 0.10285	-20.092 20.693	0.99	0.0071
C3	$\alpha = -13.992$ $\beta = 2.607$	4.337 0.7835	-3.226 3.325	0.84	0.0544
F1	$\alpha = -6.6887$ $\beta = 1.268$	0.97549 0.1762	-6.857 7.195	0.96	0.01225
F2	$\alpha = -5.9406$ $\beta = 1.1312$	1.4542 0.2627	-4.085 4.306	0.90	0.01826
F3	$\alpha = -4.2507$ $\beta = 0.8204$	1.464 0.2646	-2.902 3.101	0.82	0.01839

MSE is the Mean of Standard Errors of Regression Lines

4.3 PERFORMANCE PREDICTION OF BLENDED BINDER

One of the most important advantages of the PG asphalt system is that it can directly relate binder properties to the asphalt mixture performance. Normally three main distress predictions are needed for asphalt pavements (rutting, fatigue and low-temperature cracking). Based on the PG binder system, the $G^*/\sin\delta$ on unaged asphalt cement, the $G^*.\sin\delta$, the stiffness (S) and the m-value on aged asphalt cement are indications of susceptibility of the asphalt cement pavement to rutting, fatigue, and low-temperature cracking of asphalt pavement.

4.3.1 Rutting

Rutting or permanent deformation is plastic deformation of asphalt pavements that is load associated happening mainly at high pavement temperatures. Hicks et al. (1993) concluded that the influence of asphalt cement on rutting is highly dependent on the conditions to which the asphalt mixture is subjected. In their study, the correlation between $G^*/\sin\delta$ and the various measures of permanent deformation response were not generally strong but for some asphalt mixtures the influence of the binder was apparent. The PG binder system specifies a minimum, 1000 Pa for $G^*/\sin\delta$ at 10 rad/s. on unaged asphalt cement.

Figures 4.25 to 4.28 depict the change in log $G^*/\sin\delta$ with changes in the proportion of recycling agents. All recycling agents can satisfy the rutting criteria at a wide range of high pavement temperature and blending ratio. A higher blending ratio (lower RAP ratio) could be used when the 200-300 asphalt cement are used as a recycling agent when compared to the 300-400 asphalt cement. The binders blended with Cyclogen and Flexon give similar maximum blending ratios (RAP ratio) for the rutting criterion. The maximum of the recycling agent (minimum RAP) is significantly different for soft asphalt cements (300-400 and 200-300) comparing to the two other recycling agents (Flexon and Cyclogen). The maximum recycling agent proportion at a high

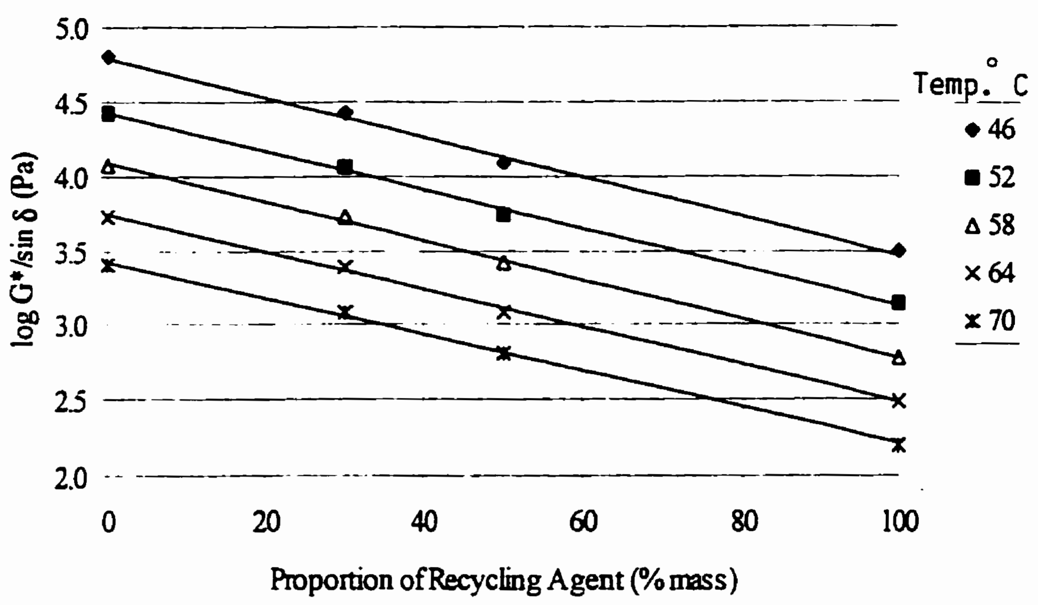


Figure 4.25 Change in the Rutting PG Criterion (G*/sin δ) with Proportion of Recycling Agents for Binders Blended with 200-300 Asphalt Cement at High Pavement Temperatures

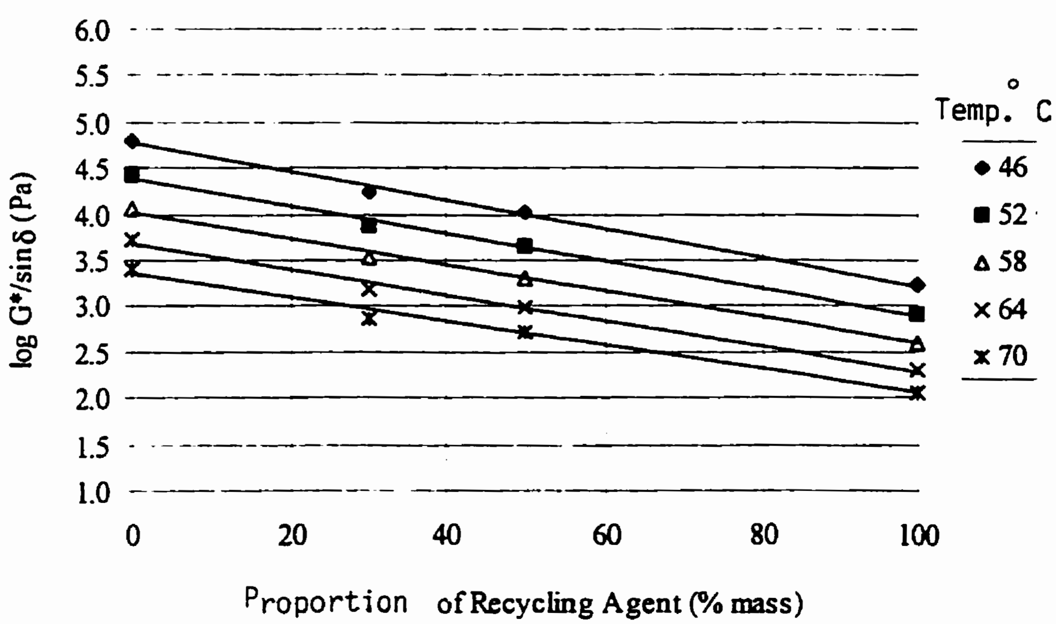


Figure 4.26 Change in the Rutting PG Criterion (G*/sin δ) with Proportion of Recycling Agents for Binders Blended with 300-400 Asphalt Cement at High Pavement Temperatures

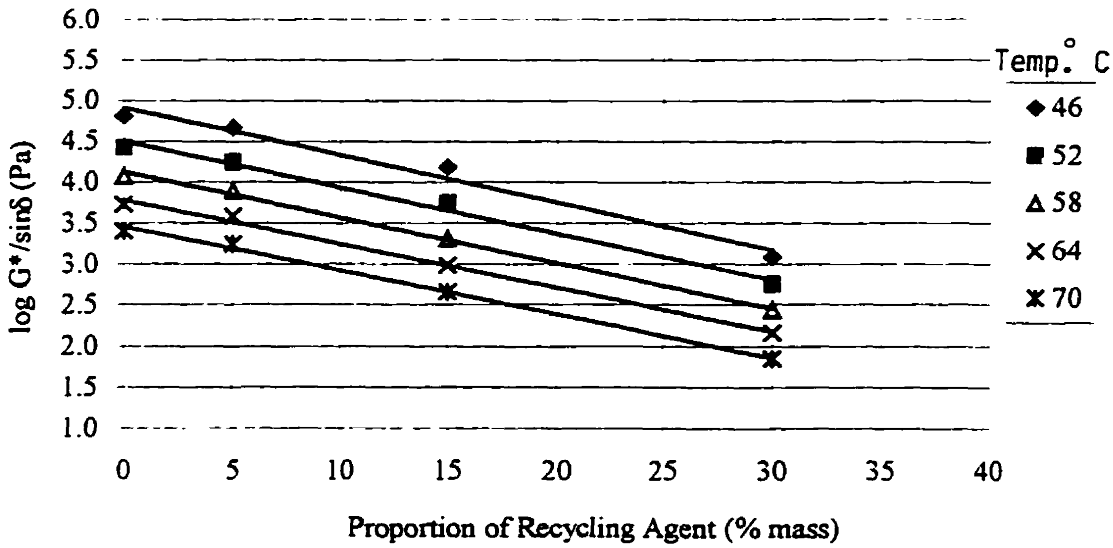


Figure 4.27 Change in the Rutting PG Criterion (G*/sin δ) with proportion of Recycling Agents for Binders Blended with Cyclogen at High Pavement Temperatures

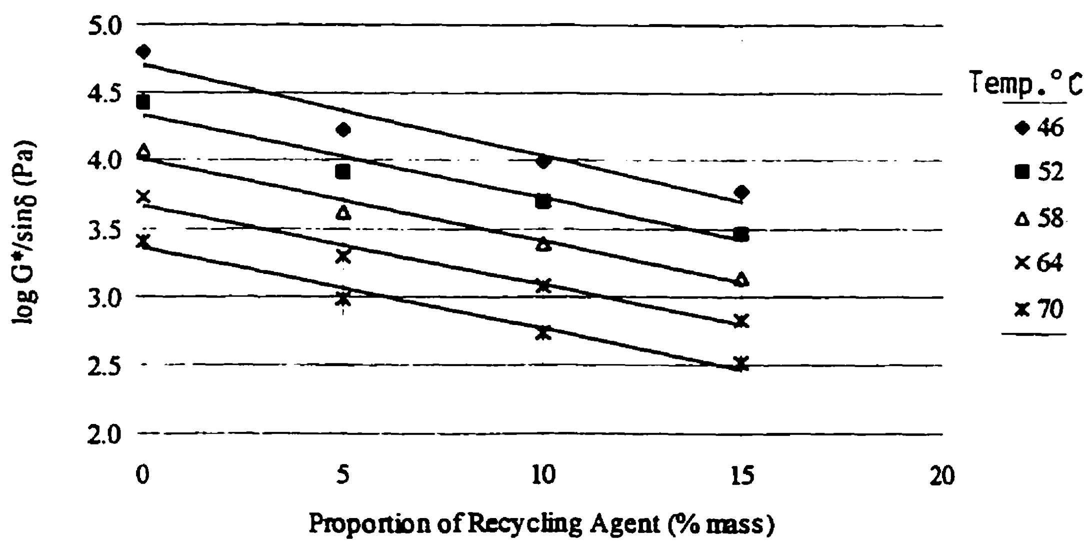


Figure 4.28 Change in the Rutting PG Criterion (G*/sin δ) with Proportion of Recycling Agents for Binders Blended with Flexon at High Pavement Temperatures

pavement temperature (70°C) were 25 and 35 percent of mass for 300-400 and 200-300 soft asphalts respectively. The maximum recycling agent proportions were six and seven percent of weight for Cyclogen and Flexon for the same pavement temperature respectively. This means that considering the rutting criterion, a higher proportion of Reclaimed Asphalt Pavement (RAP) can be used with Cyclogen and Flexon comparing to soft asphalt cements for the same high pavement temperature. This may be one reason that these types of materials (Flexon and Cyclogen) are commonly used for high RAP ratio recycling method such as hot in place recycling.

4.3.2 Fatigue

Fatigue cracking is a series of interconnecting cracks caused by fatigue failure of the asphalt concrete layer under repeated traffic loading. Hicks et al. (1993) concluded that asphalt cement properties play a critical role in the fatigue response of asphalt-aggregate mixes. However, other mixture characteristics such as air void levels, percent binder, density and aggregate characteristics can also have a significant impact on fatigue response. Therefore, asphalt cement properties alone may not provide sufficiently reliable estimates of fatigue cracking on asphalt pavement. Reese (1997) studied 24 asphalt concrete mixtures made with 12 different asphalt cements to find out the correlation between PG binder fatigue criterion and laboratory fatigue mixture properties. Reese concluded that $G^*.\sin\delta$ doesn't correlate adequately with mix fatigue tendencies. Deacon et al. (1997), in another study, emphasized that the binder alone does not determine fatigue response in the pavement structure.

In the PG system, $G^*.\sin\delta < 5000$ kPa on the aged binder is specified for fatigue cracking at intermediate pavement temperatures. Figures 4.29 to 4.32 depict the change in the PG fatigue criterion with proportions of the recycling agents used in this research At an intermediate pavement temperature of 7°C, binders blended with 300-400 could be used with a lower recycling agent proportion (68%) compared to binders blended with 200-300 recycling agent (%95). This means that a higher RAP ratio could be used with

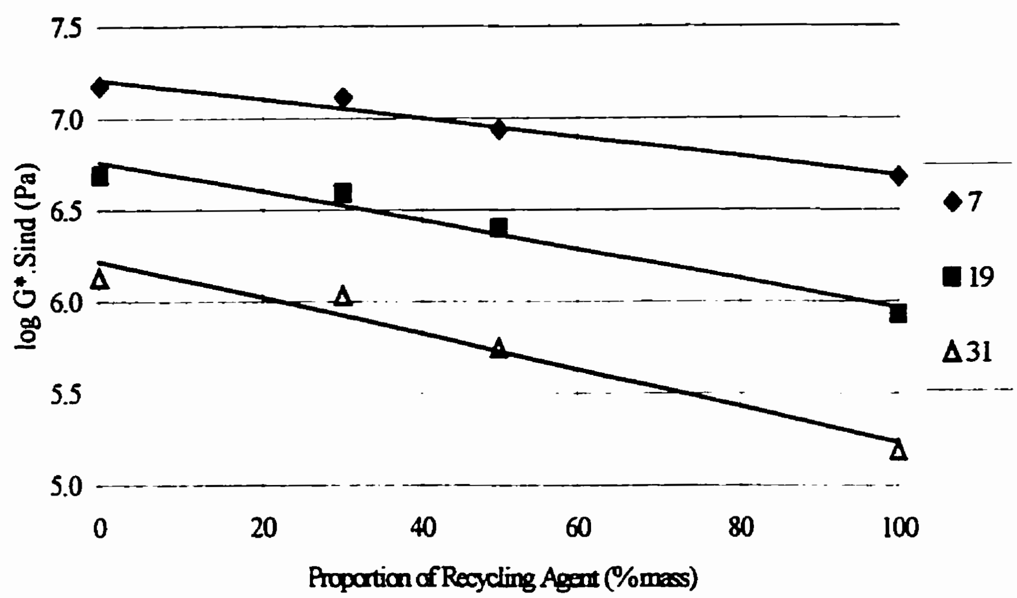


Figure 4.29 Change in the Fatigue PG Criterion (G*.sin δ) with Proportion of Recycling Agents for Binders Blended with 200-300 Asphalt Cement at Intermediate Pavement Temperatures

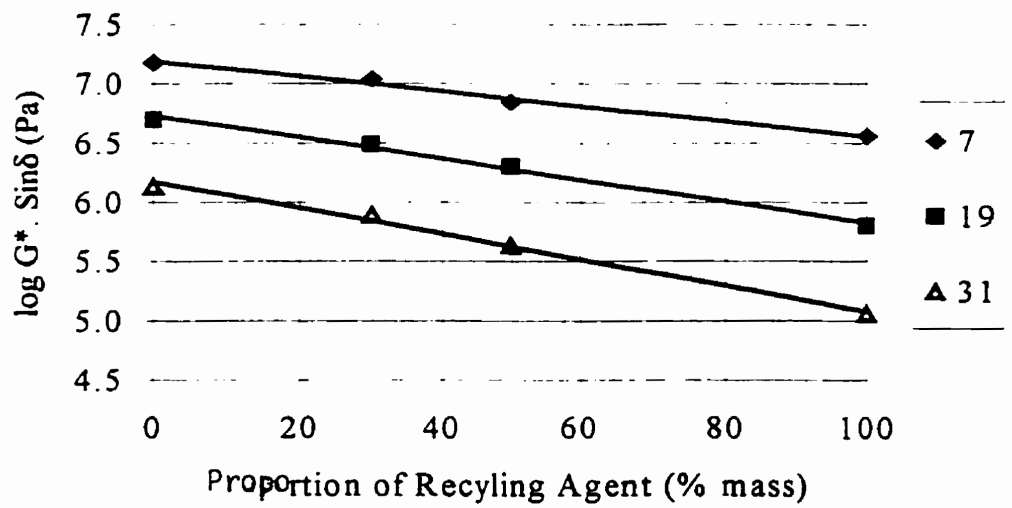


Figure 4.30 Change in the Rutting PG Criterion (G*.sin δ) with Proportion of Recycling Agents for Binders Blended with 300-400 Asphalt Cements at Intermediate Pavement Temperatures

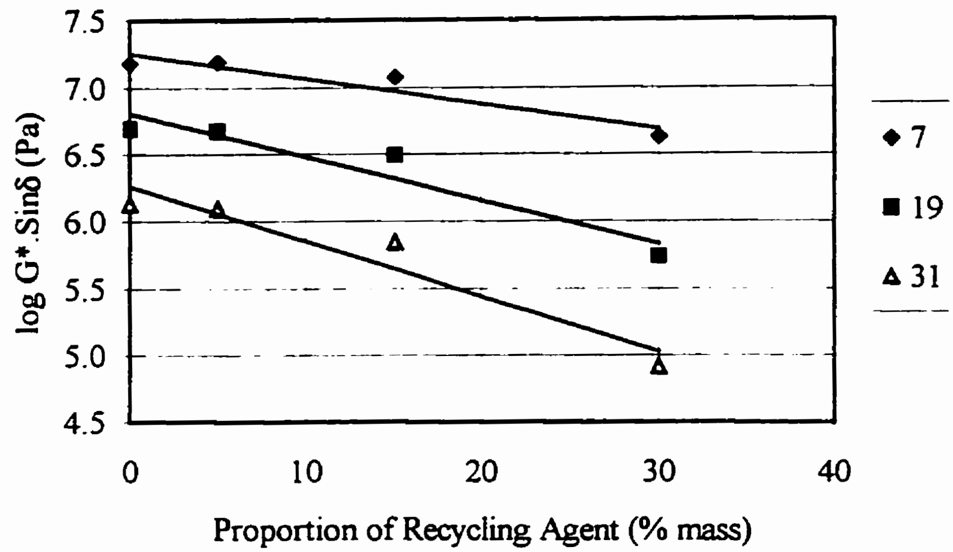


Figure 4.31 Change in the Rutting PG Criterion (G*.sin δ) with Proportion of Recycling Agents for Binders Blended with Cyclogen at Intermediate Pavement Temperatures

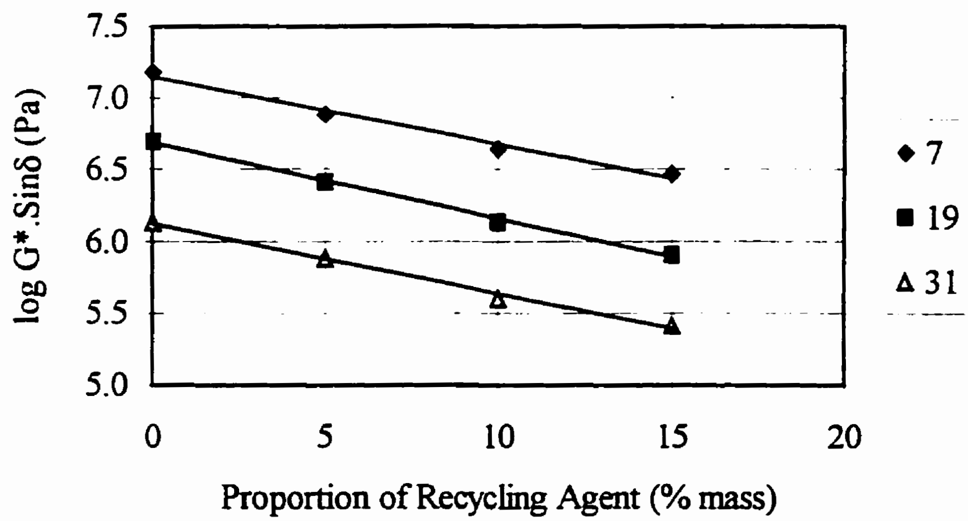


Figure 4.32 Change in the Rutting PG Criterion (G*.sin δ) with Proportion of Recycling Agents for Binders Blended with Flexon at Intermediate Pavement Temperatures

300-400 recycling agent comparing to the 200-300 asphalt cement. The low proportions of Cyclogen and Flexon (7 and 9 percent) permit the use of high ratio RAP in recycling mixtures. All recycling agents and blending proportions satisfied the fatigue PG criterion at a pavement temperature of 31°C.

4.3.3 Low-Temperature Cracking

Low-temperature cracks are mainly non-load associated cracks which initiate when the temperature in the pavement drops below a limited value that the asphalt mixture can not endure any more tensile. As discussed in Chapter 2, researchers agreed that asphalt cement is the dominant performance control component of asphalt mixtures at low pavement temperatures. The main application of the PG binder system might be predicting of non-load associated low-temperature cracking in the pavement. Hicks et al. (1993) showed that the mixture fracture temperature was highly correlated to PG low-temperature index test results (stiffness and m-value).

The PG low temperature criteria are stiffness ($S \leq 300$ MPa) and m-value ($m \geq 0.300$) on the aged asphalt cement. Changes in stiffness of blended binders versus proportion of recycling agents were graphically presented before in Figures 4.9-4.12 and changes in m-value with proportion of recycling agents presented in Figures 4.21-4.24. The figures depict that for a low pavement temperature equal to -24°C, a higher proportion of recycling agent or lower RAP ratio should be used when 200-300 asphalt is used comparing to the 300-400 asphalt cement. Higher proportion of recycling agent or lower RAP ratio should be used when Cyclogen is used instead of Flexon.

4.4 PROCEDURES FOR SELECTION OF RECYCLING AGENT

The statistical analysis in this research strongly suggests that a linear relationship can be used to predict rheological parameters of blended binder (G^*, δ, S, and m-value) based on the same parameters of aged asphalt cement and recycling agent. Two methods have been reported by Bahia et al. (1996) for selection of recycling agent based on the

PG parameters. Part of this Asphalt Institute report to the FHWA has referred to the results of this research as a Canadian study. Both methods are based on the fact that, linear models can be used for prediction of changes in rheological properties of blended binders with proportion of recycling agent (percentage of weight) of the blended binders. These two methods vary in the criteria used and are explained in more detail as follows:

4.4.1 PG Criteria Procedure

In this method, four charts are used for the selection of a recycling agent based on each PG performance criterion. The linear relationship of change in PG performance criteria with proportions of recycling agent can be used to estimate the type or the proportion of recycling agent in the recycled mixture. Three semi-logarithmic charts, for the $G^*/\sin\delta$, $G^*.\ \sin\delta$, $S(t)$, and one chart for $m(t)$ with normal scales in Y_1 and Y_2 axes can be used. Y_1 and Y_2 axes correspond to the criteria of aged asphalt cement and recycling agent. The X-axis, in all cases, is the proportion of recycling agent (percentage of weight) in the blends.

A typical graph is shown in Figure 4.33 for selection of amount of the 200-300 asphalt cement as the recycling agent. Using these graphs for design purposes, the PG performance criteria values should be measured for aged asphalt and recycling agent at the target PG temperatures for a special climate. The proportion of recycling agent can be increased or decreased until an acceptable ratio that satisfies all criteria are found.

There are some difficulties in the application of this method compared to the traditional method standardized in ASTM D 4887 (Figure 2.6, p 59). One difficulty is that there are at least four different criteria, instead of one in the viscosity and penetration method, and all of these criteria must be satisfied for the blended binder. The other difficulty is that some recycling agents are very soft and it is not possible to perform the PG binder tests on them. Some of these difficulties could be answered by the following suggestions.

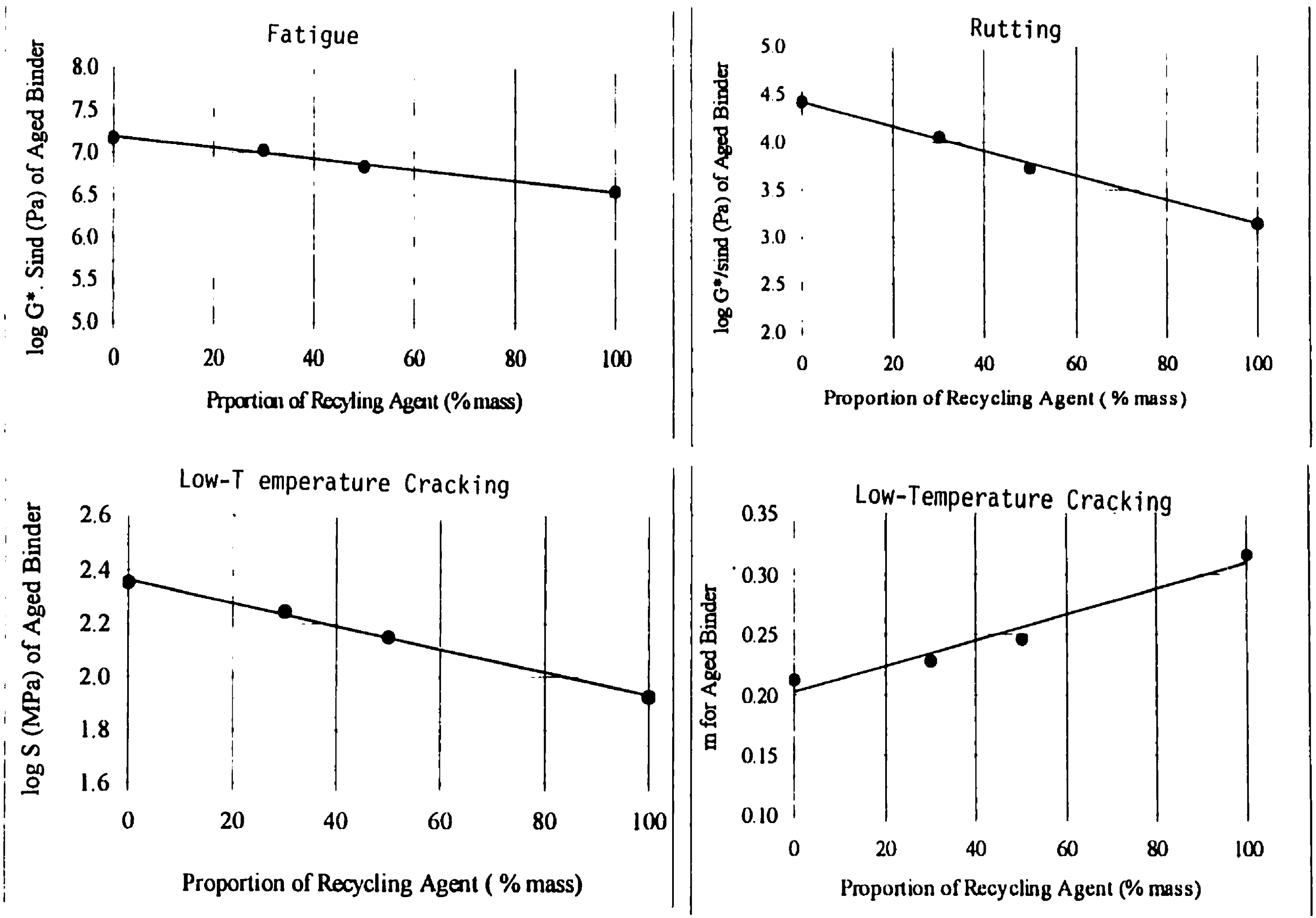


Figure 4.33 Charts for Selection of Recycling Agents Based on the PG Performance Criteria

- The PG controlling criteria could be considered flexible based on the design engineer's judgement. This means that the design engineer can select the main performance criterion that is important for a particular project. In most cases,the recycled asphalt mixtures are harder and stiffer than the virgin asphalt mixtures with similar mixture formula and are less susceptible to rutting. Therefore satisfying the rutting criteria in the PG binder system may not be essential for recycled mixtures. This assumption, which applies for most cases, can reduce the necessary controlling criteria.

- As discussed before, asphalt cement is the dominant component, which controls transverse cracks in the asphalt mixtures at low pavement temperatures. This is especially important for the Canadian situation where transverse cracks are the main concern of the highway agencies. Therefore, for these situations, controlling the low pavement criteria (S and m-value) for blended binders are more important and should be considered.

- If the recycling agent is too soft to perform the PG binder tests on them, the results on a blend of recycling agent and aged binder can be used instead of the PG criteria for recycling agent. This has been used in this research for Cyclogen and Flexon.

- The proportion obtained from the above method can be checked with the result obtained from traditional method (viscosity chart ASTM D 4887) as a control

4.4.2 PG Temperature Procedure

In this procedure, one graph is used to select the type and the ratio of recycling agent based on the linear relationship for change in the PG temperature grading of the aged and recycling agent. In this graph, Y1 and Y2 axis depict the temperatures at which the aged asphalt cement and recycling agent satisfied the PG performance criteria and the

X axis is the proportion of recycling agent.

There is a high correlation between satisfied temperatures for stiffness (S) and m-value criteria. Therefore, two temperature lines ($G^*/\sin\delta$ and S or m-value) are enough for this purpose. In the PG system, asphalts are graded with high and low- pavement temperatures and there is not any number related to intermediate pavement temperatures. Therefore, the fatigue associated line ($G^*. \sin\delta$) is not necessary to be incorporated in this graph and two lines represents the change in high and low grades (pavement temperatures) of blended binders at different proportions. The following procedure can be used to estimate the effect of RAP on the grade of new asphalt to be used in a recycled mixture.

1. Extract and recover the asphalt from RAP and estimate the PG grade for aged asphalt cement (high and low performance temperature). This will give two points on the Y_1 of the graph.

2. Test the recycling agent (soft asphalt) with the PG method and estimate its PG grade (high and low performance temperature). If the recycling agent is very soft, a blend of recycling agent and aged asphalt cement could be used for this purpose. The high and low grade of recycling agent or blend will give two points on the Y2 axis or any blended ratio.

3. Draw a line between high and low temperature points. These lines represent the change in the PG grade for any proportion of recycling agent in the blends.

4. The proportion of recycling agent or the PG grade (high and low satisfied pavement temperatures) can be estimated for various blended binders.

An example of this type of graph for binders blended with 300-400 asphalt cement is shown in Figure 4.34. The same graph can be easily constructed for other recycling agents. The advantage of this graph is that for any proportion of recycling agent the PG temperatures grading can be estimated. The disadvantage of this method is that it can not These methods have several advantages compared to the traditional method for selection of recycling agent, which is based on viscosity/ penetration of aged asphalt and recycling agent. The most important advantage is that they can be used for selection of recycling agent to control low-temperature cracks for cold climate conditions give the best recycling agent proportion for satisfying the fatigue criterion in the PG.

4.4.3 A Case Study

The Performance Graded (PG) asphalt binder specifications are based on performance criteria at various temperatures that correspond to the local climate. The procedure for selecting a PG asphalt binder for a particular climatic region is as follows:

- Determine the maximum ambient air temperature for the area with a corresponding reliability.
- Determine the maximum pavement temperature corresponding to the ambient air temperature based on an algorithm from SUPERPAVE™ software.
- Determine the minimum ambient air temperature for the area with a corresponding reliability.
- The minimum pavement temperature is assumed equal to the minimum ambient air temperature.

A study in Canada (C-SHRP 1995) showed that low pavement temperatures in the PG binder system, which is equal to the minimum air temperature, is too conservative and is not consistent with experience in regions with very low winter temperatures. This indicates that pavement temperatures are significantly higher than air temperatures in severe cold weather conditions. Until this issue is settled, C-SHRP suggests that it is not necessary to select a reliability of more than 50% for low pavement temperature.

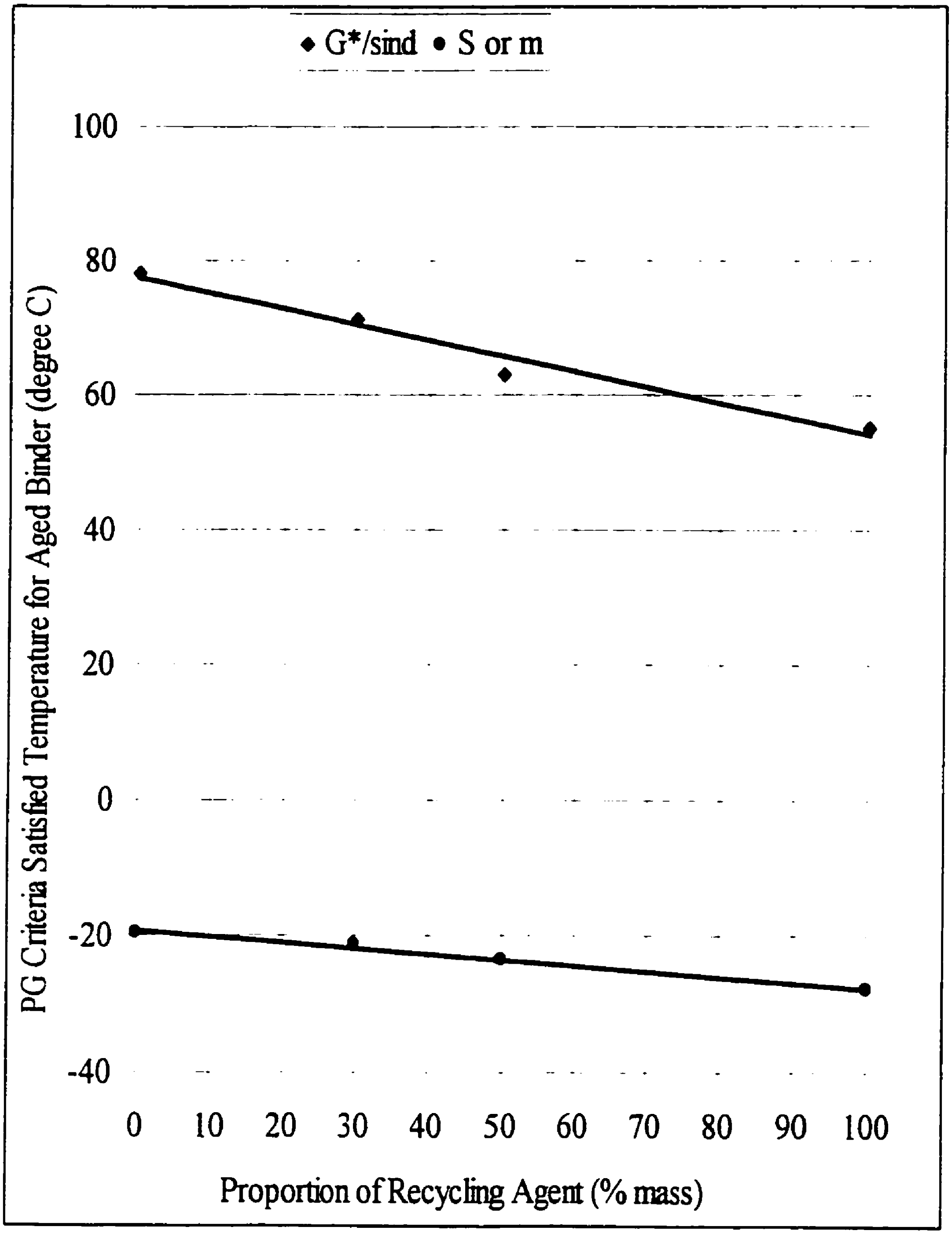


Figure 4.34 PG Temperature Criteria Chart for Selection of Recycling Agent (300-400 Asphalt Cement)

To show the capability of the PG system for selection of a performance-based recycling agent, a high and low pavement temperature were selected. Table 4.9 summarizes the SHRP weather database for the maximum and minimum pavement temperatures for the Saskatoon area. The maximum and minimum pavement temperatures will specify the most appropriate PG asphalt binder for any climate conditions. Table 4.10, shows the proper PG asphalt cement binder for Saskatoon area. The temperatures at which the PG asphalt tests should be satisfied are specified based on the grading of asphalt cement. Table 4.11 summarizes the temperatures that each PG binder test should be satisfied for Saskatoon area.

Based on the SHRP weather database and the PG air and pavement algorithm 52-40 and 58-52 PG binders are performance graded binder with 50 and 98 percent reliability for Saskatoon climatic conditions. Using discussed methods for selection of recycling agents it can be concluded that using a soft asphalt binder such as 300-400 can satisfy the PG rutting criterion with high reliability but it can not warrant the low-temperature cracking even with 50% reliability (Soleymani et al. 1995).

It can be concluded that, based on the PG system, the only way to satisfy the Saskatoon climate low-temperature condition is to use modified asphalt cements. This conclusion is not in agreement with another study (Haas, 1970) which showed a 150-200 asphalt cement from Lioyminster for Taxi "A" at the Saskatoon Airport had an excellent low temperature performance, no cracks, after nine years of services. This shows that the PG system need some modification for implementation for cold climate conditions such as in Saskatoon.

Table 4.9 Maximum and Minimum Pavement Temperatures for Saskatoon (Source: SHRP Weather Database)

Reliability %	Max. Air Temp. °C	Max. Pav. Temp. °C	Min. Air Temp. °C	Min. Pav. Temp. °C
50	+34	+51	-39	-39
98	+38	+54	-47	-47

Table 4.10 Performance Grade (PG) Asphalt Binders for Saskatoon

Reliability %	PG
50	52-40
98	58-52

Table 4.11 Criteria Testing Temperatures for PG Asphalt Binder for The Saskatoon Area

PG	DSR (Unaged)	DSR (RTFO)	DSR (PAV)	BBR (PAV)	DTT (PAV)
52-40	52°C	52°C	10°C	-30°C	-30°C
58-52	58°C	58°C	7°C	-42°C	-42°C

4.5 DISCUSSION

In the analysis of data in this chapter, it was clear, that the coefficients of determination were lower with binders blended with Flexon and Cyclogen compared to the binders blended with soft asphalts in all cases. This could be attributed to the blending high and low viscosity materials when Flexon and Cyclogen were used. This conclusion is in agreement with Chaffin et al. (1995) that showed the viscosity of some blended binders can be predicted better with the Irving model [4.2] instead of the Arrhenius model [4.1]. Chaffin et al. showed that the interaction parameter, G_{12}, in the Irving model, is very important for blends with very different components properties but it was close to zero when two asphalt cements blended with each other. In order to verify this conclusion with the PG parameters, it is necessary to study the effect of the G_{12} parameter in the future studies in this area.

4.6 SUMMARY

A linear and a non-linear model were compared for change in complex shear modulus, phase angle, stiffness, and m-value of blended binders with proportion of recycling agent in the blends (percentage of mass). The statistical analysis showed that for all PG testing parameters (G^*, δ, S, and m-value), the non-linear model has no significant advantage to the linear model. Therefore, a linear model is proposed for change in log G^*, log S, δ, and m-value with proportion of recycling agent in the blend.

The temperature dependency of the complex shear modulus and the stiffness of blended binders with change in proportion of recycling agent were studied. The statistical analysis showed that a shift value could be used for this purpose. This means that a shift value, which is specific for each blended binder, can be used to predict the log G^* and log S of blended binders at other temperatures than the measured one.

The temperature dependency of phase angle and m-value concluded that a logarithmic model is accurate enough for prediction of δ and m-value with temperature for all blended binders.

Two methods for selection of the type and proportion of recycling agent for asphalt pavement recycling projects were evaluated. In the first method, four charts are used for different PG performance criteria ($G^*.\sin\delta$, $G^*/\sin\delta$, S, and m-value). There are some difficulties in the application of this method because of trial and error for satisfying all criteria. The second method uses one chart based on temperatures that were satisfied by the PG performance criteria. This method is easier to use and can give the best proportion and/or recycling agent based on the PG asphalt grade for specific climate conditions.

In a case study, the application of developed models and design methods for selection of the best recycling agent for Saskatoon climate conditions were carried out. It concluded that by selecting of soft asphalt cement such as 300-400, the reliability for low-temperature cracking was less than 50%. This conclusion is not in agreement with some other studies and shows that the PG system needs some modifications for implementation in cold climate conditions.

It should be noted that all conclusions and analyses in this chapter were based on the PG test data at specification loading times or frequencies. This means that all DSR and BBR testing data and their related analysis were at 10 rad./s. and 60 seconds respectively. Characterization of blended binders at other frequencies or loading times are studied in Chapter Five with their master curves.

The correlation of linear relationships, for PG parameters, were lower when Flexon and Cyclogen were used compared to when soft asphalts were used. This could be related to high viscosity differences between aged asphalt cement and recycling agent.

It is necessary to study the effect of interaction parameter, G_{12}, in the Irving model when high and low viscosity materials are blended.

CHAPTER FIVE

MASTER CURVES FOR BLENDED BINDERS

5.1 INTRODUCTION

To characterize the rheological behavior of asphalt cement, the temperature and loading time (or frequency) dependencies of complex shear modulus, stiffness, and phase angle should be studied. Chapter Four studied the blended binders with the PG specification, which means the dynamic shear rheometer and bending beam rheometer results were considered at 10 rad/s. and 60 seconds respectively. This chapter describes the temperature and frequency dependencies of blended binders using master curves.

A FORTRAN computer program (SHIFT) is used to build the master curves. The rheological SHRP A-002A model of asphalt cement, which is a hyperbolic equation, is accepted as the best available characterization model (SHRP A-369). The SPSS statistical computer program is used to estimate the parameters of this hyperbolic model for all blended binders. The relationship between master curve parameters and proportion of recycling agent is studied. The temperature dependency of blended binders is studied with the defining temperature (T_d) in the WLF Equation.

5.2 BUILDING MASTER CURVES

To have a complete explanation of a linear viscoelastic material behavior, such as asphalt cement, it should be characterized for a wide range of temperatures and loading times (frequencies). Master curves have been used by many researchers (Brodnyan 1958, Sisko and Brunstrum 1968, Jongepier and Kuilman 1969, Dobson 1969, Duthle 1972, Dickinson and Witt 1974, Maccarroni 1987, Goodrich 1988 and 1991, and Christensen 1992) for characterization of asphalt cements. Characterization of asphalt cement with master curves has not been used widely for practical purposes because of difficulties in developing an accurate model for loading time and the time consuming process for building the master curve.

Researchers have proposed different models to explain the behavior of asphalt cement for a wide range of loading times. A review of these models and their advantages and disadvantages were explained in Chapter Two. The SHRP A-002A reviewed previous models and concluded a hyperbolic equation was applicable for loading time dependency of complex shear modulus of asphalt cements. This model is accurate and more practical than the previous models proposed for asphalt cement. Using the computer for analyzing complex engineering problems has created an opportunity to simplify the process in building master curves.

To build the modulus master curve for blended binders, complex shear modulus results (at 10 rad./s.) were used. Because the complex shear moduli at other frequencies were not available, it was decided that they could be estimated using the SHRP A-002A models. For this purpose Equations [5.1], [5.2], and [5.3] were used respectively.

$$R = (\log 2) \log \left[G^*(\omega) / G_g \right] / \log(1 - \delta(\omega) / 90) \qquad [5.1]$$

$$\omega_c = \omega \left\{ \left[\frac{G_g}{G^*_{(\omega)}} \right]^{\log 2 / R} - 1 \right\}^{R / \log 2} \qquad [5.2]$$

$$G^*(\omega) = G_g\left[1 + (\omega_c/\omega)^{(\log 2)/R}\right]^{-R/\log 2} \qquad [5.3]$$

where:

R = the rheological index

$G^*(\omega)$ = complex shear modulus, in Pa, at frequency ω, rad/s.

G_g = glass modulus, typically 1 GPa

δ = the phase angle, degree

ω_c = the crossover frequency, rad/s.

A typical calculation of complex shear modulus for a wide range of frequencies for aged 150-200 asphalt cement is presented in Table 5.1. Figure 5.1 depicts complex shear modulus of aged 150-200 asphalt cement in a log-log scale versus frequencies at high and intermediate pavement temperatures

The method of *"reduced variable"* or *"time-temperature superposition"*, which was discussed in Chapter Two, was used to shift the shear modulus on to a base temperature. According to this method, the effects of temperature and frequency for viscoelastic material are completely separable. (Ferry 1980). Use of *time temperature superposition* method involves collecting data such as complex shear modulus over a range of temperature and frequencies and shifting the collected data, such as Figure 5.1, parallel to the frequency axis to form a smooth and continuous function.

The amount of shifting required in this process is quantified through the shift factor, *a(T)*. The change in *a(T)* , or *log a(T)* with temperature indicates the temperature dependency of a given material. Figure 5.2 illustrates the concept of *time-temperature superposition* method for calculating shift factors and building master curves for aged 150-200 asphalt cement.

Table 5.1 Estimated Shear Modulus for Aged 150-200 Asphalt Cement at Different Frequencies with the SHRP A-002A Model

TEMP. °C	*G*(Pa)* at log ω= 1 rad/s.	δ at ω=10 rad./s.	*R*	ω_c log rad./s.	*G*(Pa)* at log ω= -3 rad/s.	*G*(Pa)* at log ω= -2 rad/s.	*G*(Pa)* at log ω= -1 rad/s.	*G*(Pa)* at log ω= 0 rad/s.	*G*(Pa)* at log ω=2 rad/s.	*G*(Pa)* at log =3 rad/s.
7	18274380	39.10	2.11	1.569188	77189.29	393374.9	1698464	6119468	45367533	94776214
13	8798750	45.48	2.02	11.52563	19456.28	114960.7	583898.8	2492359	25530788	61187243
19	3212160	50.88	2.07	61.08816	4604.792	29511.23	165738.7	796591.8	10730262	29606156
25	1369060	55.48	2.07	261.4571	1351.166	9301.186	56980.28	303142.1	5156927	16050856
31	583450	59.03	2.10	901.4896	443.6048	3196.286	20757.24	118667.2	2418076	8333530
37	256600	62.28	2.11	2935.999	153.7639	1155.289	7912.509	48295.27	1161141	4399372
46	58860	67.23	2.13	21497.24	24.87504	198.3653	1466.937	9857.963	305157.5	1345306
52	25030	71.44	2.02	84772.07	7.687563	65.00922	516.8371	3788.788	145294.2	722511.2
58	11320	75.60	1.87	298931.9	2.570946	22.92713	194.9332	1552.303	73588.89	413998.7
64	5190	78.53	1.78	860721.1	0.971509	8.937846	79.19134	665.2618	36627.51	226708.9
70	2470	81.08	1.68	2258924	0.393787	3.713106	34.01707	298.7867	18748.91	126677

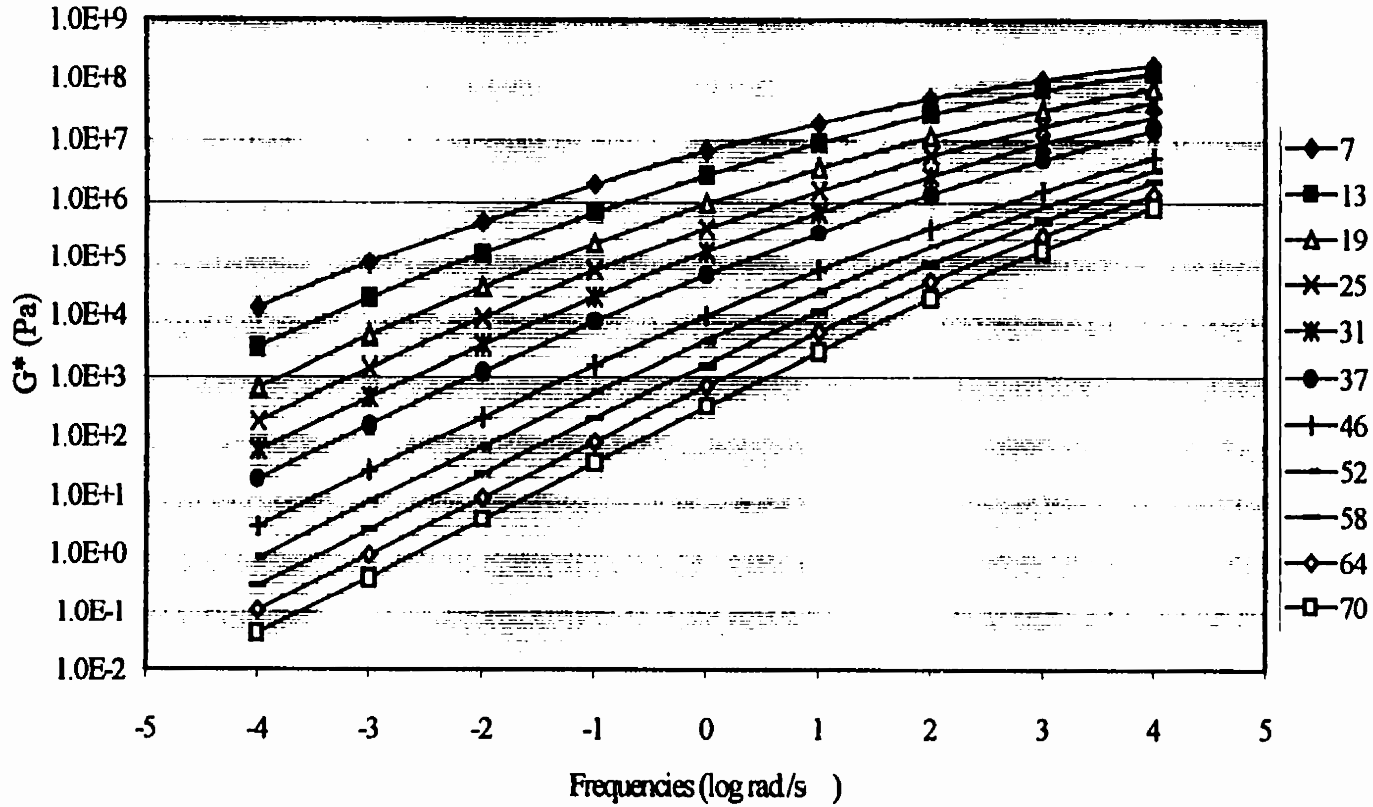


Figure 5.1 Change in Complex Shear Modulus of Aged 150-200 Asphalt Cement Versus Frequencies at High and Intermediate Pavement Temperatures

Either construction of the master curve, using the method of reduced variable, is done manually or using specially designed software. The manual procedure is arbitrary in nature and lacks a standard criterion for obtaining the best fit of the master curve. The computerized procedures involves selecting few points at the end of the isothermal modulus curves and using different statistical procedures (such as least square method) to match the end parts of the neighboring curves and estimate the required shift. Such procedures are adopted in many software packages that are built into different existing dynamic analysis methods. The problem with such procedures is that they rely heavily on the few end data points without considering the overall behavior of the response (Bahia 1993).

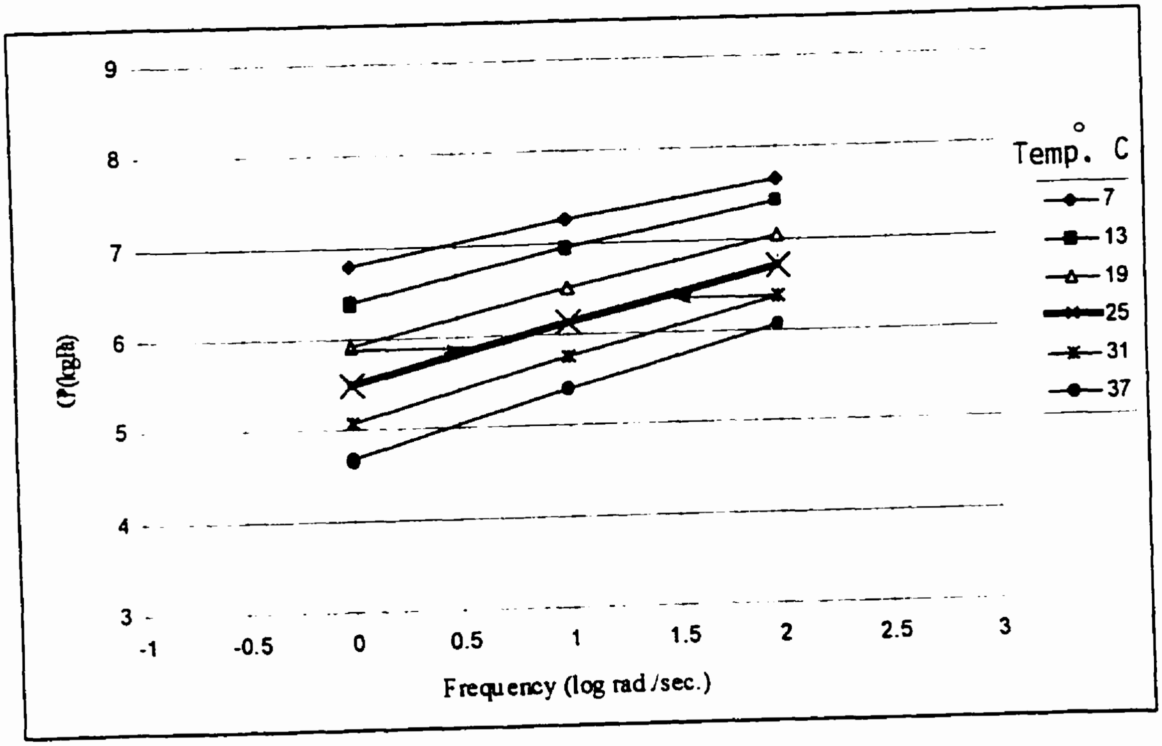


Figure 5.2 Illustration of Time–Temperature Superposition Method for Aged 150-200 Asphalt Cement

In this research, estimated complex shear moduli between 1 and 100 rad/s. frequencies (or between 0-2 log of frequencies) were used for calculating shift factors and building master curves. This is a practical range of frequencies in the applications of binders for pavements, which simulate the speed of vehicles in a range between zero to 100 km/h (Brown 1973). The complex shear modulus at 25°C was considered as the base temperature and complex shear moduli at other temperatures were shifted to this temperature.

A FORTRAN computer program (named SHIFT) was developed for calculating the shift factors and building the master curves. This program with a typical input and output is presented in Appendix B. This computer program uses the Newton-Raphson method for calculating the roots of equation [5.1] for each G^*. The Newton-Raphson method requires the evaluation of both the function $f(x)$, and the derivative $f'(x)$, for each estimated x. The Newton-Raphson method consists, geometrically, of extending the tangent line at a current point x_i until it crosses zero, then setting the next guess x_{i+1} to the abscissa of that zero-crossing. The most important advantage of the Newton-Raphson method is its fast convergence for cases whose derivative can be evaluated efficiently, and whose derivative is continuous and non-zero in the neighborhood of a root. (Press et al. 1992).

To calculate shift factors with SHIFT, the SHRP rheological model equation [5.3] was transformed into a log scale. Therefore, the $f(x)$ and $f'(x)$ were used as follows in this program:

$$f(x) = log\, G^*(\omega) = log\, G_g - \frac{1}{log\, 2/R}\, log\left[1 + (\omega_c / \omega)^{(log\, 2)/R}\right] \qquad [5.4]$$

$$f'(x) = \frac{d(G^*)}{d(\omega)} = \frac{1}{\omega} \times \frac{1}{1 + \left(\frac{\omega}{\omega_c}\right)^{log 2/R}} \qquad [5.5]$$

where the parameters were explained before.

The value of shift factors, *log a(T)*, were calculated using SHIFT Program for all virgin and aged blended binders and are presented in Tables 5.2 and 5.3. To compare the shift factor functions of the studied blended binders, change in shift factors with temperatures, for virgin and aged blended binders, are shown in Figures 5.3 and 5.4. The shift factors show a smooth and appear typical for all blended binders relationship with temperatures within the range of testing temperatures for virgin and aged blended binders. Figure 5.5 to 5.14 depict the effect of temperature with shift factors for each aged and unaged blended binder. The results indicated that blended binders are similar in their temperature dependency in the range of temperature studied. This is in agreement with other researchers who studied asphalt cements at low pavement temperatures (Bahia 1992).

For all blended binders in this study, the aged shift factors were higher than the unaged shift factors at temperatures higher than base temperature (25°C) and were less at temperatures lower than base temperatures. Binders blended with Flexon had higher differences between aged and unaged shift factors. This means that these binders (F1, F2, and F3) were more temperature dependent compared to other blended binders.

5.3 TEMPERATURE DEPENDENCY OF SHIFT FACTOR

The temperature dependency of the shift factor can be explained by Williams, Landel, and Ferry Equation (WLF, 1955). The WLF Equation is described by the following relationship:

$$\log a(T)_d = [-C_1 (T - T_d)] / [C_2 + (T - T_d)] \qquad [5.6]$$

where:

$a(T)_d$ = the horizontal shift factor relative to the defining temperature, T_d

T = the test temperature in °C or °K

T_d = the defining temperature from which the data is shifted, in °C or °K

C_1 and C_2 = empirically determined constants

Table 5.2 Calculated Shift Factor Values for Virgin Blended Binders

Binders / Temp. °C	M1	M2	N1	N2	C1	C2	C3	F1	F2	F3
7	2.0573	1.7982	2.0106	1.9215	2.2634	2.0585	1.7948	1.5294	1.6523	1.6242
13	1.3154	1.1986	1.3346	1.1913	1.4713	1.2801	1.1502	1.0046	1.0525	1.1038
19	0.6223	0.5245	0.5703	0.5324	0.7264	0.5553	0.5565	0.4799	0.5373	0.5609
31	-0.5186	-0.4745	-0.5406	-0.5115	-0.5201	-0.5547	-0.5064	-0.3857	-0.4341	-0.5136
37	-1.0223	-1.0137	-1.0319	-0.9794	-1.0396	-1.0302	-0.8942	-0.8300	-0.9069	-1.0243
45	-1.7754	-1.7428	-1.7403	-1.7117	-1.8281	-1.7375	-1.5113	-1.4612	-1.5179	-1.6432

Table 5.3 Calculated Shift Factor Values for Aged Blended Binders

Binders / Temp. °C	M1	M2	N1	N2	C1	C2	C3	F1	F2	F3
7	2.1641	2.1025	2.2477	2.1497	2.2931	2.2032	1.9670	2.1554	2.1243	2.6688
13	1.4037	1.4391	1.4904	1.3775	1.4452	1.4030	1.2210	1.3877	1.3436	1.6316
19	0.6736	0.7122	0.6877	0.6845	0.6781	0.6684	0.5592	0.6746	0.6486	0.7158
31	-0.5840	-0.6341	-0.5894	-0.6750	-0.6550	-0.6381	-0.5504	-0.6452	-0.6379	-0.7310
37	-1.1655	-1.2489	-1.2566	-1.2827	-1.2405	-1.2210	-1.0788	-1.2546	-1.1887	-1.4002
45	-2.0709	-2.1000	-2.2136	-2.2094	-2.1236	-2.1253	-1.8233	-1.9487	-1.8885	-2.3012

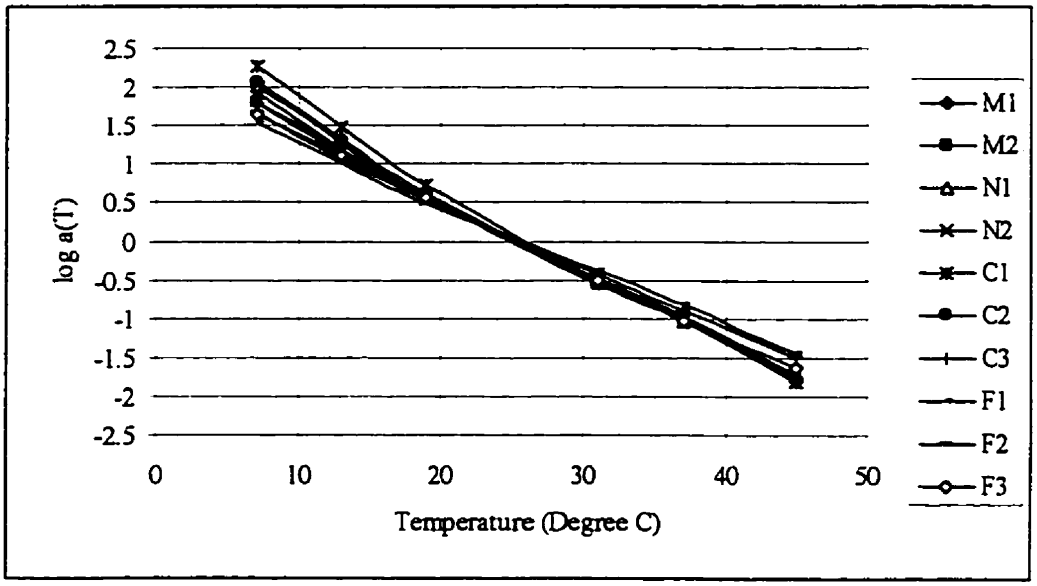


Figure 5.3 Effect of Temperatures on Shift Factors for Unaged Blended Binders

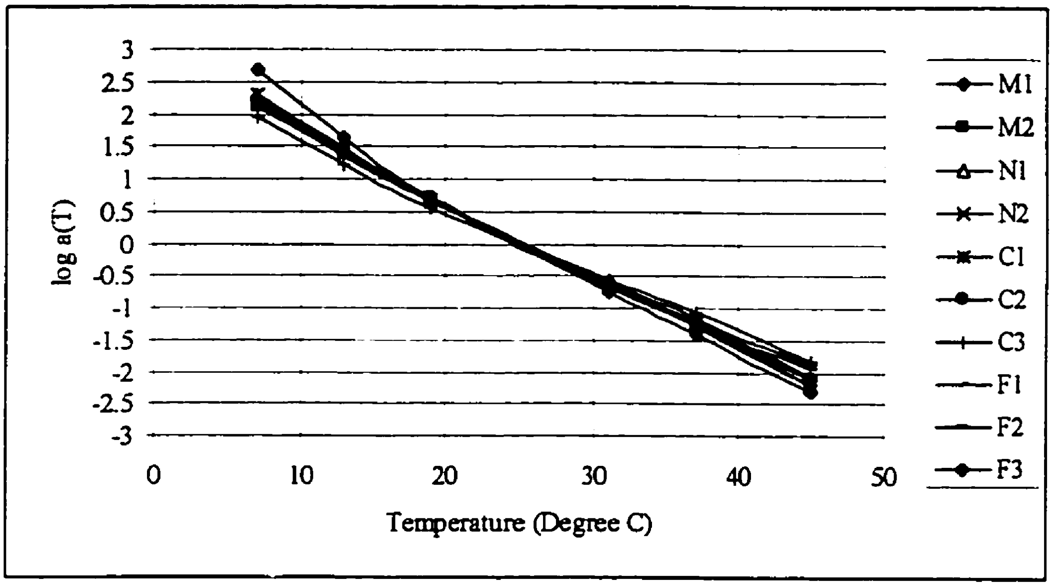


Figure 5.4 Effect of Temperatures on Shift Factors for Aged Blended Binders

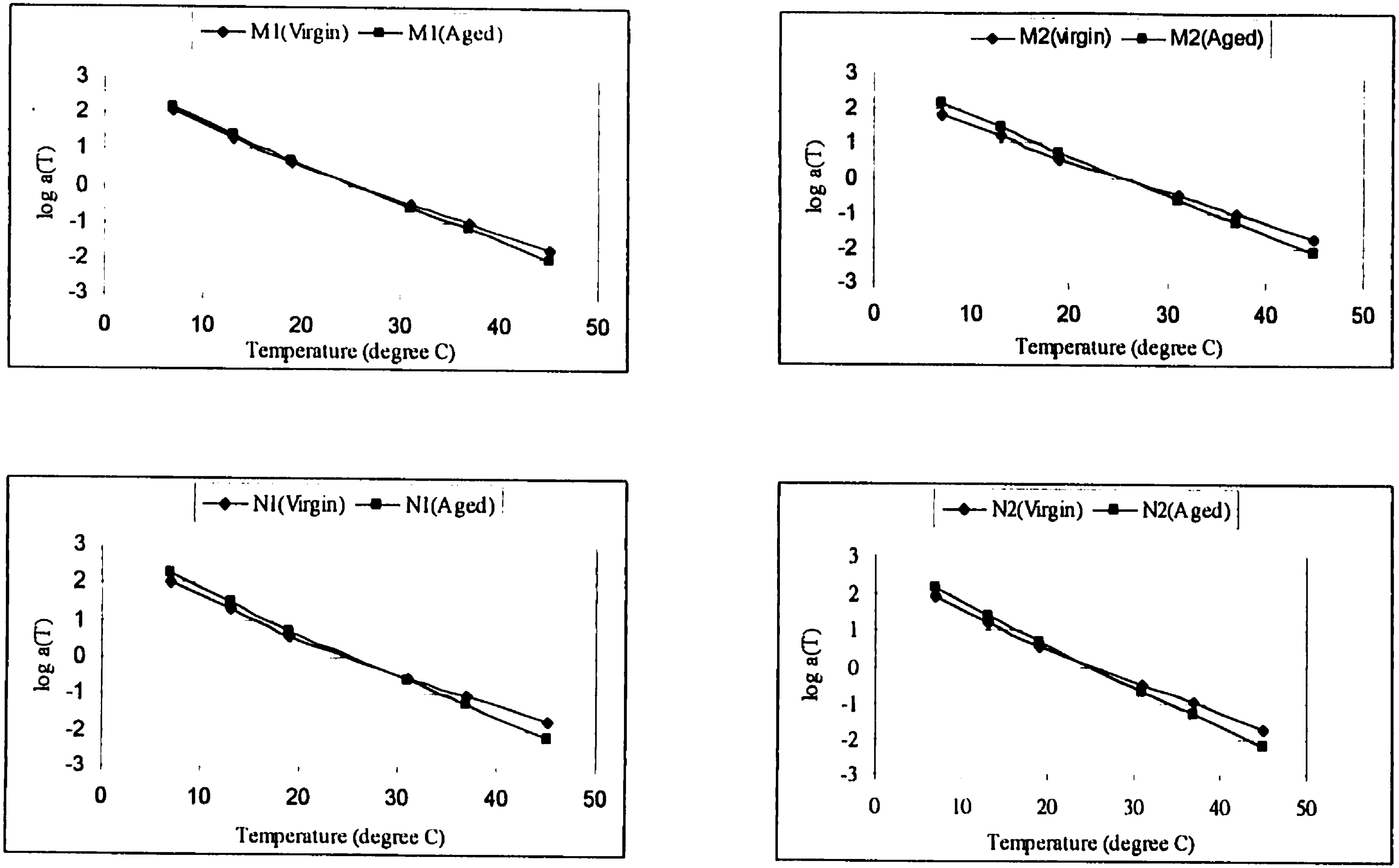
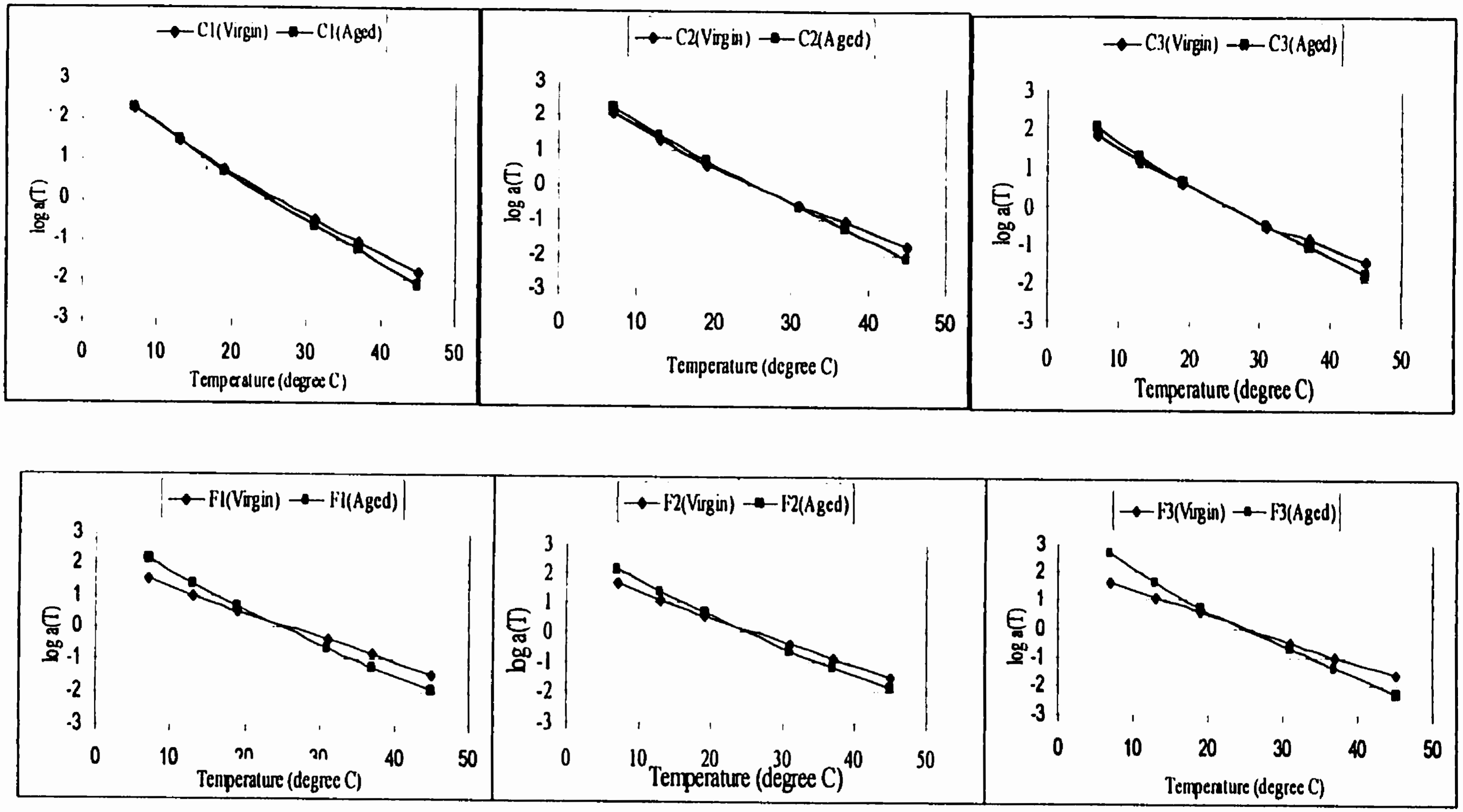


Figure 5.5, 5.6, 5.7, and 5.8 Effect of Temperatures on Shift Factors of Virgin and Aged Binders Blended With 200-300 and 300-400 Asphalt Cement



Figures 5.9, 5.10, 5.11, 5.12, 5.13, and 5.14 Effect of Temperatures on Shift factors for Virgin and Aged Binders Blended with Cyclogen and Flexon

For paving asphalt cements, various researchers (Dobson 1969, Jongepier and Kuilman 1969, Brodnayan et al. 1960) have reported different constant parameters on the over prediction of $a(T)$ when using the WLF Equation. Table 5.7 presents values for parameters (C_1 and C_2) used by researchers.

Table 5.4 Various Suggested Values for Parameters in the WLF Equation

Researchers	C_1	C_2
Jongepier and Kuilman (1969)	8.86	101.6
Dobson(for low- temp.) (1969)	12.5	142.5
Dobson(for high- temp.) (1969)	8.86	101.6
Maccaroni (1987)	22.68	230
Sisko (1968)	28.6	292
SHRP A-002A (1993)	19	92

The defining temperature, which is asphalt specific, can be calculated with considering C_1 and C_2 from one of the above studies and using the trial and error or non-linear least squares methods. In this study, the shift factors were calculated previously relative to complex shear modulus at 25 °C (Table 5.2 and 5.3). Therefore, the WLF Equation was modified as below:

$$log\, a(T/T_r) = \frac{-19\times(T-T_d)}{(92+T-T_d)} + \frac{19\times(T_r-T_d)}{(92+T_r-T_d)} \qquad [5.7]$$

where:

T_r is the reference temperature for calculated shift factors, in case of this study 25°C.

The physical significant of the defining temperatures (T_d) is dependent on concept such as glass transition and free volume. Many researchers believe T_d is related to the glass transition temperature independently. The T_d is a characteristic temperature, which describe or defines the temperature dependency of asphalt. As the value of T_d decreases, the temperature dependency of asphalt cement will decrease. Christensen (1992) showed, for eight SHRP core asphalts, defining temperatures tend to be 5 to 10 °C higher than the glass transition measured using dilatometry.

The importance of glassy transition temperature, T_G , is that the coefficient of thermal expansion of asphalt cement changes in the vicinity of this temperature. Therefore any indirect method for estimating T_G is a useful tool for characterization of asphalt cement at low temperatures.

In solving equation [5.7], there is a system with one unknown parameter, T_d, one equation and several observations at different temperatures. The value of T_d were calculated by the least squares method and are presented in Table 5.5 for all unaged and aged blended binders.

The defining temperature ranged between –1.0 to –34.7 C for all aged and unaged binders used in this study. The defining temperatures decreased after aging for all binder. This means that aged binders are less temperature dependent than the unaged binders. The unaged binders blended with Flexon had the lowest defining temperatures, which indicate the high temperature dependency of these blended binders. This conclusion is in agreement with the graphical comparison of change in shift factors with temperatures for the same binders shown in Figures 5.12, 5.13, and 5.14.

The relationship between defining temperatures (T_d) and proportion of recycling agents in the blended binders was studied. Figure 5.15 to 5.18 depict the change in the T_d with proportion of the recycling agents in the blends when 200-300, 300-400, Cyclogen, and Flexon were used as recycling agents.

Table 5.5 Calculated Defining Temperature (T_d) from WLF Equation for all binders

Binders	T_d (°C) Unaged	T_d (°C) Aged
150-200	-23.9	-12.3
200-300	-22.2	-18.2
300-400	-25.5	-18.7
N1	-17.9	-8.0
N2	-21.8	-7.5
M1	-15.7	-10.6
M2	-25.4	-8.2
C1	-10.1	-7.6
C2	-18.1	-8.8
C3	-25.2	-17.9
F1	-34.7	-9.7
F2	-29.5	-11.6
F3	-24.5	-1.0

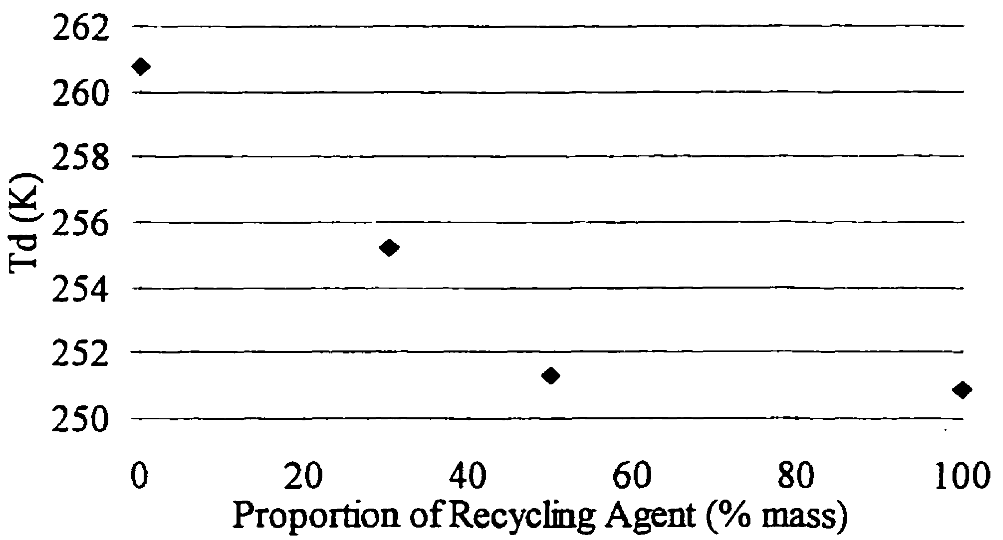


Figure 5.15 Change in T_d with Proportion of Recycling Agent (200-300 Asphalt Cement) in the Blends

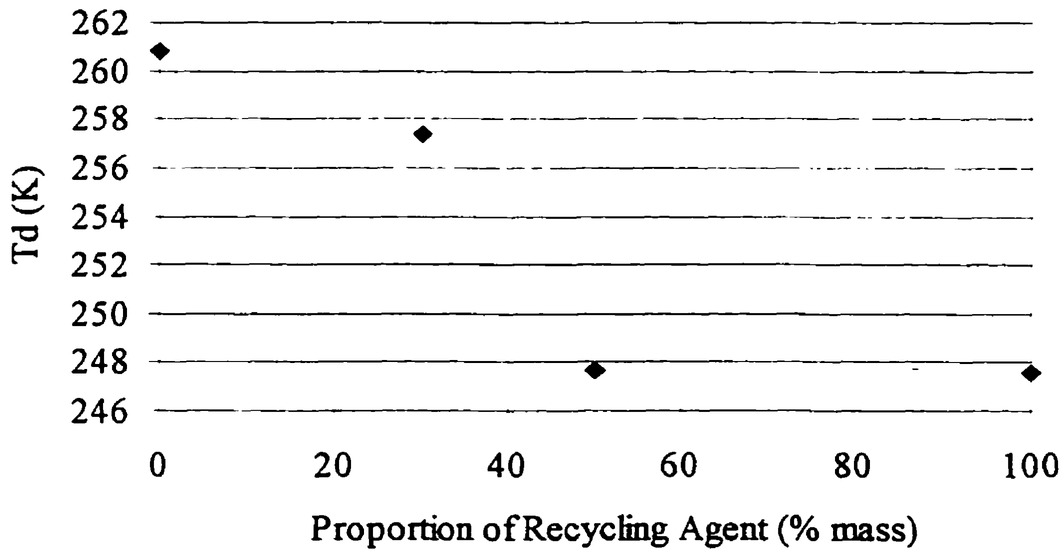


Figure 5.16 Change in T_d with Proportion of Recycling Agent (300-400 Asphalt Cement) in the Blends

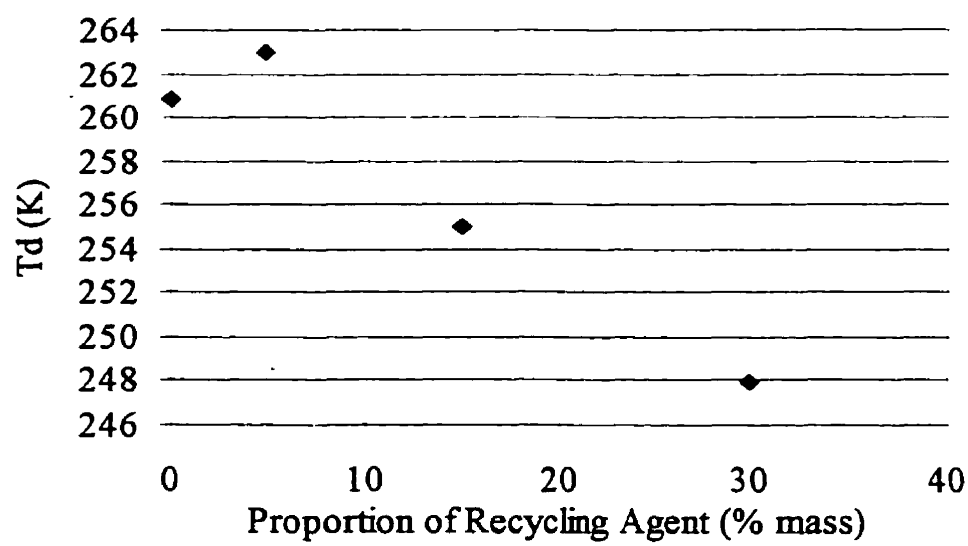


Figure 5.17 Change in T_d with Proportion of Recycling Agent (200-300 Asphalt Cement) in the Blends

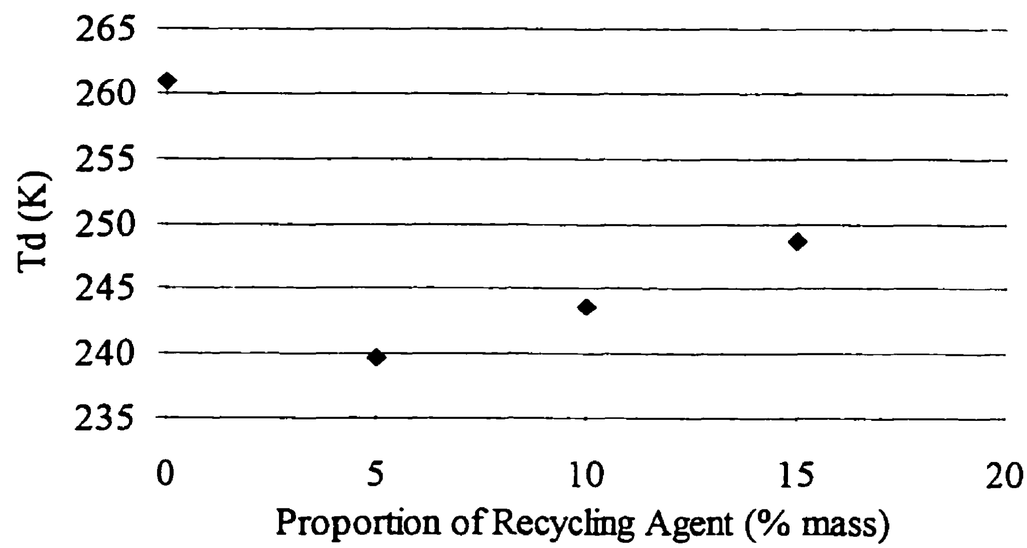


Figure 5.18 Change in T_d with Proportion of Recycling Agent (200-300 Asphalt Cement) in the Blends

For all recycling agents used in this study, the T_d decreased as the proportion of recycling agent in the blend increased. This indicates that blended binders with lower recycling agent are less temperature dependent. A exponential equation can explain the change in the T_d with proportion of recycling agent.

5.4 CHARACTERIZATION OF BINDERS WITH MASTER CURVE PARAMETERS

The master curve describes the behavior of asphalt cement at a wide range of frequencies and temperatures. Normally a master curve depicts the change of complex shear modulus, phase angle, or stiffness with loading times or frequencies on a log-log scale. At high frequencies, the shear modulus approaches a limiting value, which is the glass modulus in shear. The glass modulus physically represents the limiting modulus attained at high frequencies or short loading times and low temperatures. The characteristic value of 10^9 Pa in shear is typical for a wide range of organic materials (Ferry, 1980). At low frequencies, the slope of the log-log plot of shear modulus versus frequency approaches 1:1, which signifies that viscous flow has been reached and the asphalt is behaving as a Newtonian fluid. Based on the SHRP A-00A2 study the master curve for each asphalt could be specified using a hyperbolic equation [5.3] with two parameters: rheological index (R) and crossover frequency (ω_c).

The rheological index (R) is the difference of the log of glassy modulus to the log of complex modulus at the crossover frequency. As the R increases, the master curve will become more flat, indicating a more gradual transition from purely elastic behavior to steady- state flow. A high rheological index also means delayed elastic behavior. As the glassy modulus is approached, the response of a given asphalt cement becomes purely elastic; as steady-state flow is approached, the response of a given binder becomes purely viscous. At intermediate frequencies or temperatures, the bulk of the response is delayed inelastic in nature. Also R could show the energy storage of asphalt cement during deformation. Asphalt cements with higher R-values will store more energy during deformation than asphalts with lower R-values, at similar modulus or compliance levels.

The crossover frequency, ω_c, physically represents the frequency at which the phase angle is 45°, or alternately, where the storage and loss modulus are equal (Anderson et al 1991). Jonegepier and Kuilman (1968) and Dickinson and Witt (1974), pointed out that the point at which $\delta(\omega)$=45° occurs approximately at the point where the viscous asymptote and glassy modulus cross. The crossover frequency can be thought of as a location parameter for the master curve, and is indicative of the overall hardness of a given asphalt. The lower the crossover frequency at a given temperature, the harder will be the asphalt.

In a master curve, the low frequencies corresponded to long loading times or high temperature behavior of asphalt cement. For all asphalt cements at high loading frequencies, the complex shear modulus declines to a constant value, which is the glass modulus and can be assumed to be 1 GPa.

Non-linear least squares method was employed to estimate the hyperbolic function parameters (R, ω_c) of blended binders. The SPSS computer package, version 6.1.2, was used to run the statistical regression. This program estimated the parameters using the Levenberg-Marquardt method. Tables 5.6 and 5.7 show the estimated parameters from non-linear regression analysis for virgin and aged blended binders. Included in each table are the estimated parameter, the standard error of estimate for the parameter and associated degree of freedom. The coefficients of determination (R^2) values were, in all cases, higher than 0.9998. This is the case when modeling a smooth well-behaved function. However, the real reason for high values of R^2 is the use of calculated shear modulus with SHRP model instead of measuring them in the lab. Therefore, the R^2 values are not meaningful, and have not been included in the Tables 5.4 and 5.5.

Table 5.6 Estimated Rheological Indices (*R*) for Asphalt Cements

Binders	*R* (Virgin)	Std. Error	*R* (Aged)	Std. Error	*R* Aging Ratio
150-200	1.51	0.0065	2.07	0.0127	1.37
200-300	1.46	0.0086	2.09	0.0034	1.43
300-400	1.43	0.0082	2.11	0.0067	1.47
N1	1.99	0.0126	2.67	0.0105	1.34
N2	1.84	0.0132	2.65	0.0112	1.44
M1	2.02	0.0092	2.52	0.0017	1.25
M2	1.84	0.0039	2.61	0.0031	1.42
C1	2.14	0.1192	2.63	0.0084	1.23
C2	1.87	0.0140	2.47	0.0100	1.32
C3	1.46	0.0109	1.76	0.0155	1.20
F1	1.56	0.0090	3.12	0.00175	2.00
F2	1.54	0.0225	3.10	0.0109	2.01
F3	1.04	0.0171	2.70	0.0127	2.60

Degree of freedom = 20

Table 5.7 Estimated Crossover Frequencies (ω_c) for Asphalt Cements

Binders	ω_c Virgin	Std. Error	ω_c Aged	Std. Error	ω_c Aging Ratio
150-200	4.36	2.46	0.48	0.18	0.1101
200-300	4.87	3.04	3.16	1.18	0.6488
300-400	5.16	3.25	3.34	1.65	0.6472
N1	3.22	1.79	0.72	0.25	0.2236
N2	3.76	2.30	1.24	0.83	0.3298
M1	3.37	1.80	1.31	0.15	0.3887
M2	3.81	1.81	1.58	0.48	0.4147
C1	2.68	1.29	0.61	0.16	0.2276
C2	3.61	3.49	1.43	0.05	0.3961
C3	5.27	1.84	3.93	2.5	0.7457
F1	3.38	2.40	0.37	1.25	0.1095
F2	3.56	2.10	0.48	0.09	0.1348
F3	3.35	2.10	0.88	0.18	0.2626

Degree of freedom = 20

For all aged blended binders, rheological indices (R) were higher than the unaged R for the same blended binders. This means that the master curves for aged binders were flatter than the same virgin binders. This can also be seen graphically from Figures 5.19 to 5. 31. For all asphalt cements and blended binders, shear modulus increased as binders aged, this is expected. The difference between modulus of aged and unaged binders decreased as the frequencies increased.

Based on estimated crossover frequencies (ω_c) for blended binders, in Table 5.7, the C2 blended binder is the hardest one. Binders blended with Flexon showed higher ω_c aging ratio. This suggests that these binders were harder than other binders after aging and therefore are more susceptible to cracking at low temperatures.

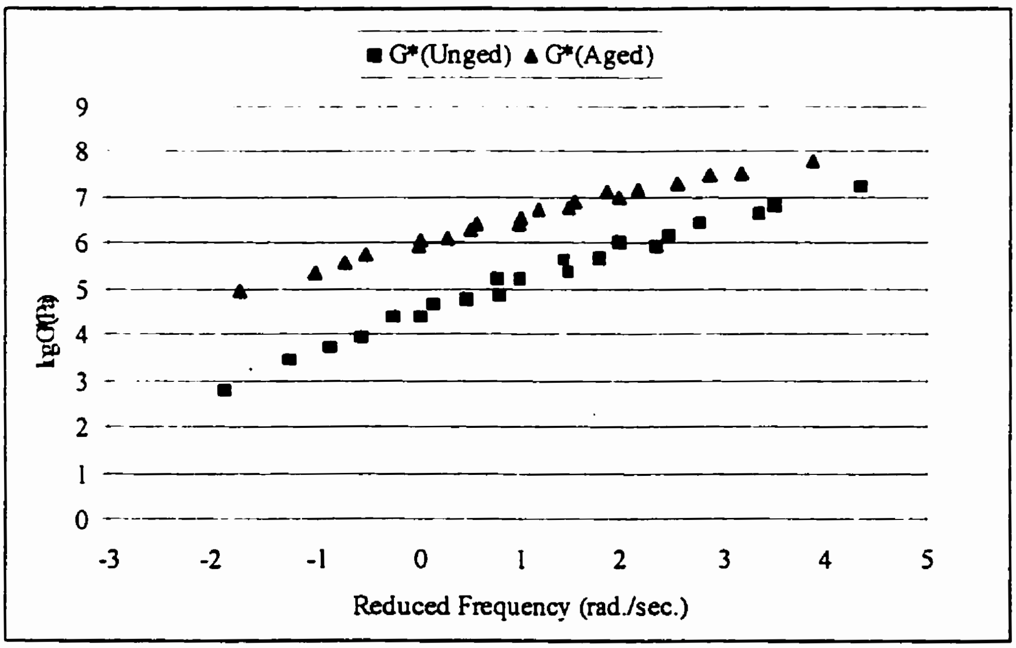


Figure 5.19 Master Curves for Unaged and Aged 150-200 Asphalt Cement

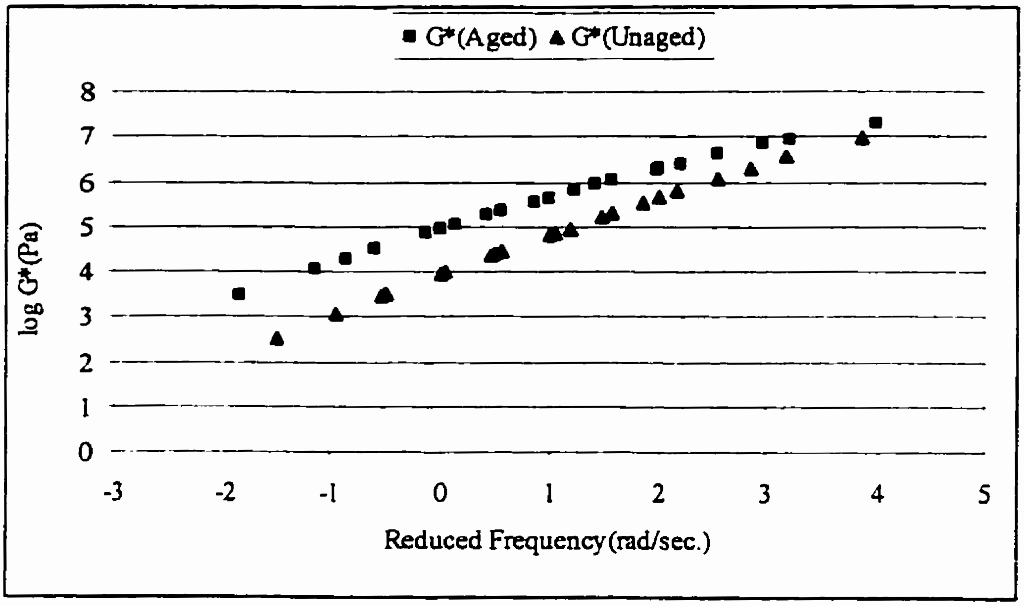


Figure 5.20 Master Curves for Unaged and Aged of 200-300 Asphalt Cement

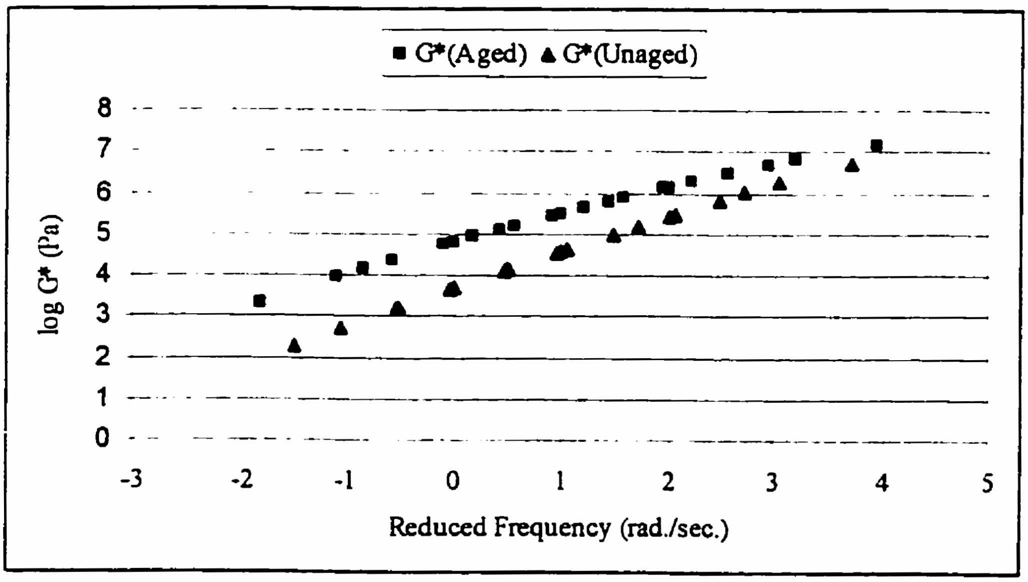


Figure 5.21 Master Curves for Unaged and Aged of 300-400 Asphalt Cement

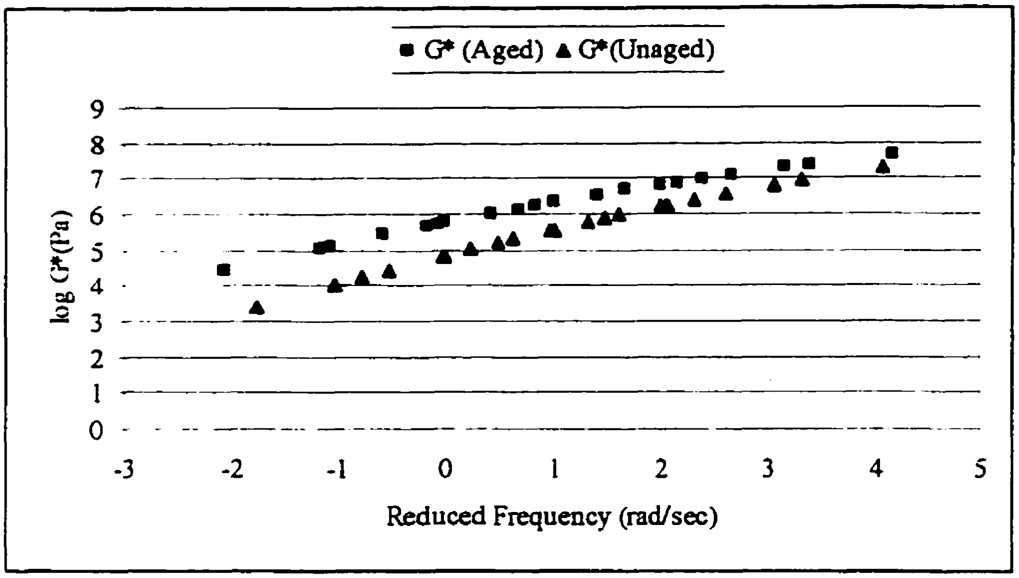


Figure 5.22 Master Curves for Unaged and Aged of M1 Blended Binder

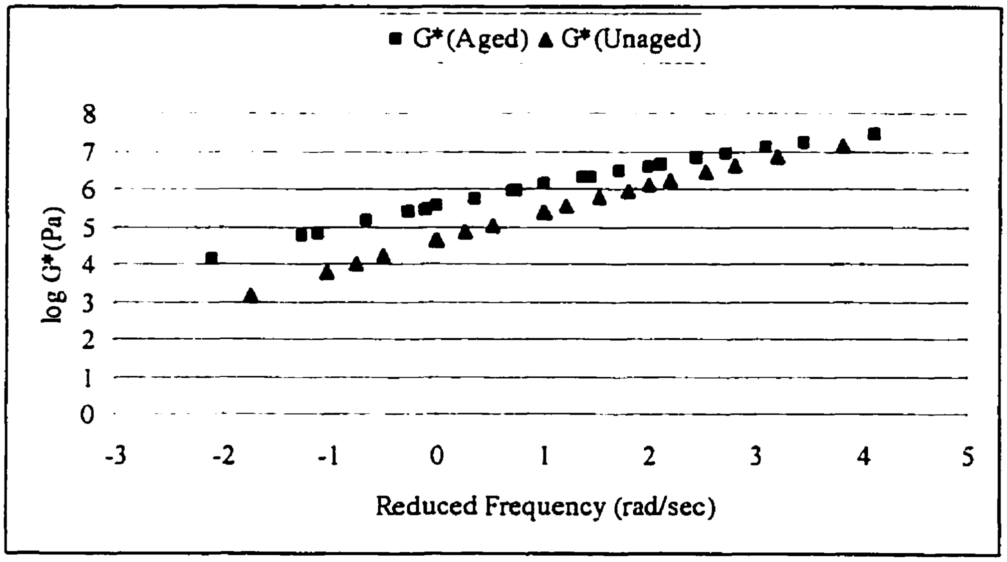


Figure 5. 23Master Curves for Unaged and Aged of M2 Blended Binder

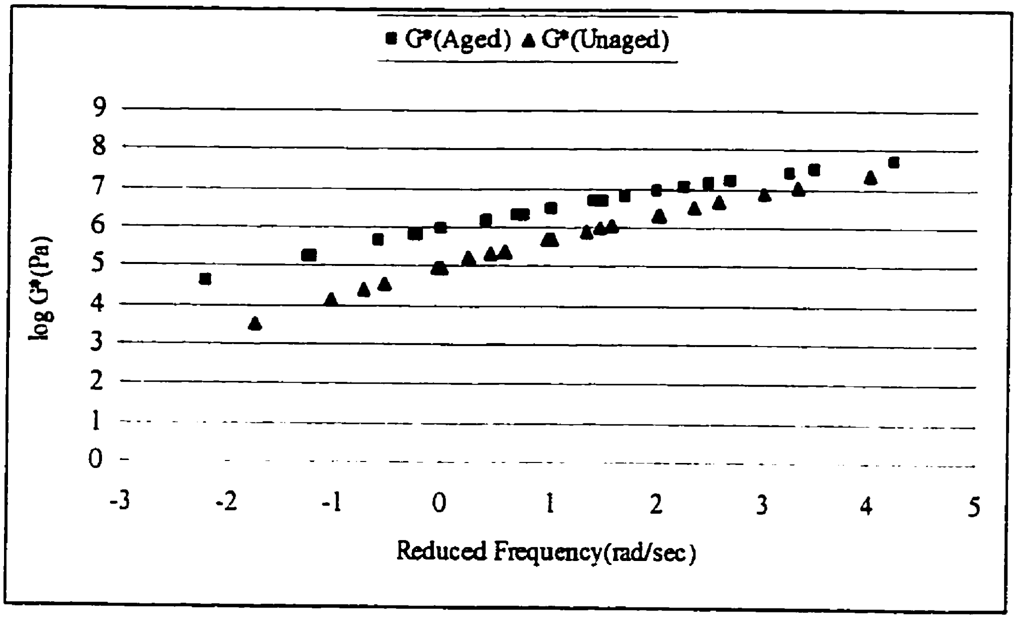


Figure 5.24 Master Curves for Unaged and Aged of N1 Blended Binder

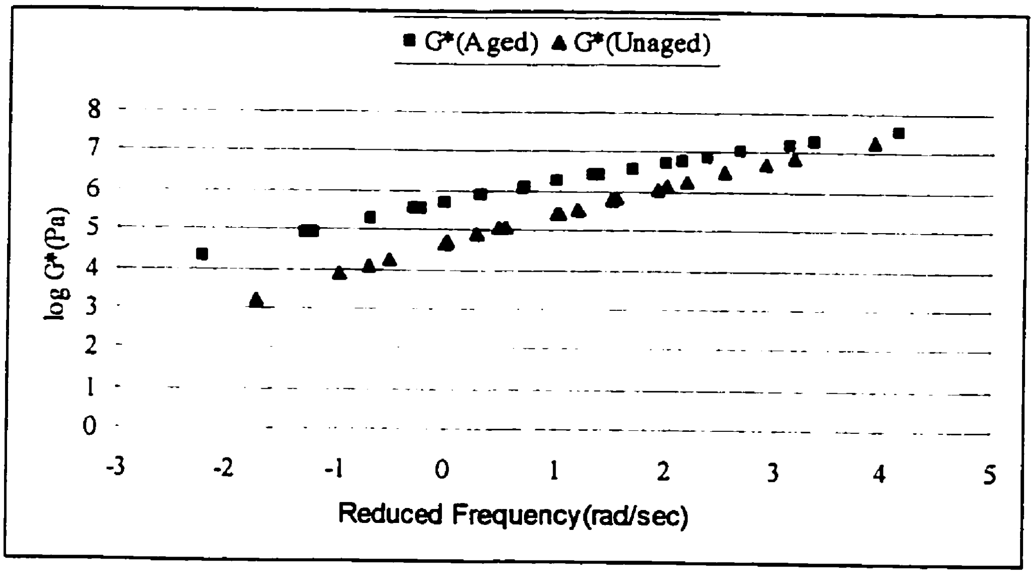


Figure 5.25 Master Curves for Unaged and Aged of N2 Blended Binder

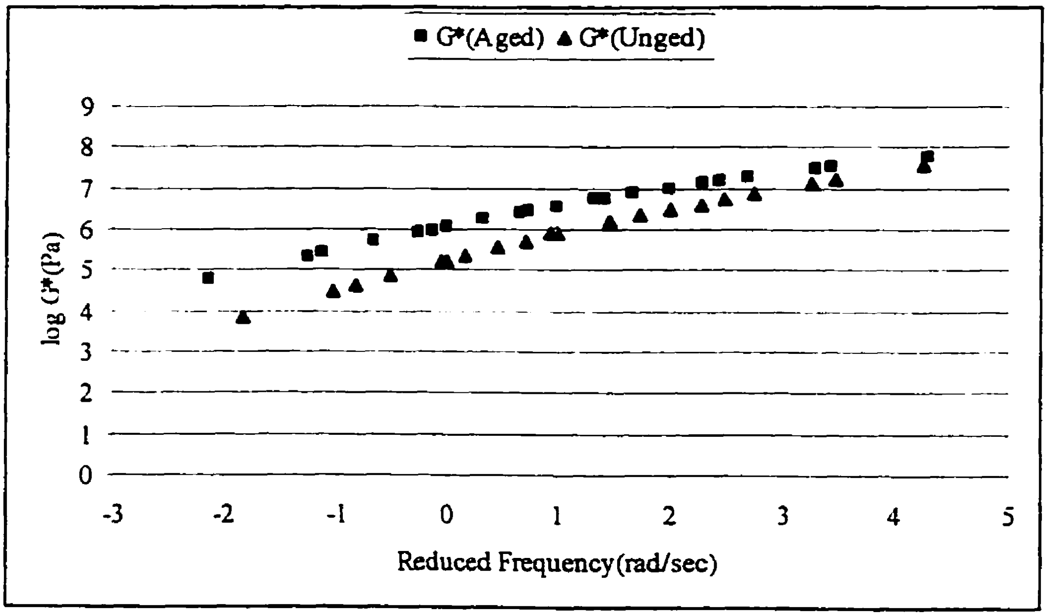


Figure 5.26 Master Curves for Unaged and Aged of C1 Blended Binder

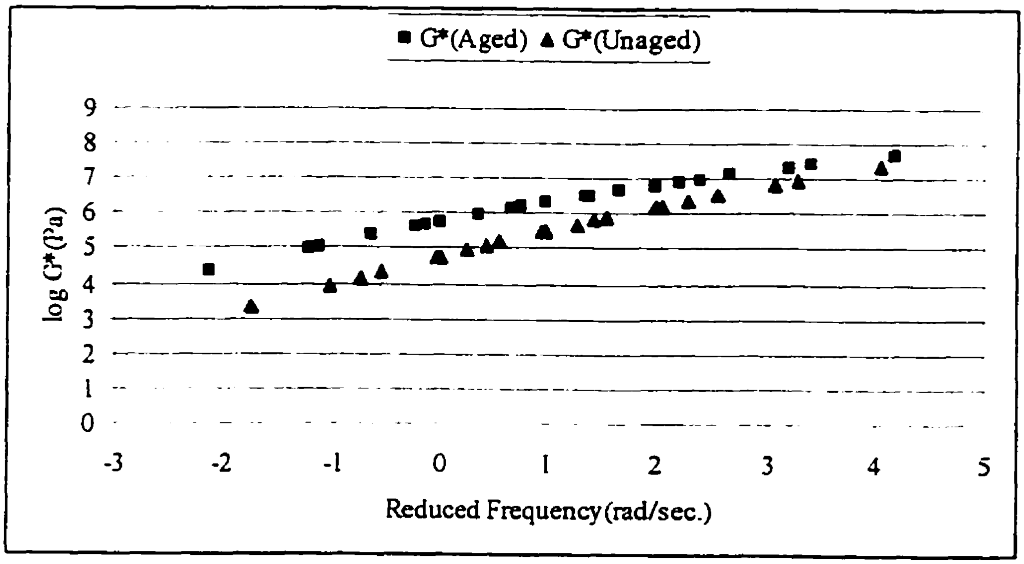


Figure 5. 27 Master Curves for Unaged and Aged of C2 Blended Binder

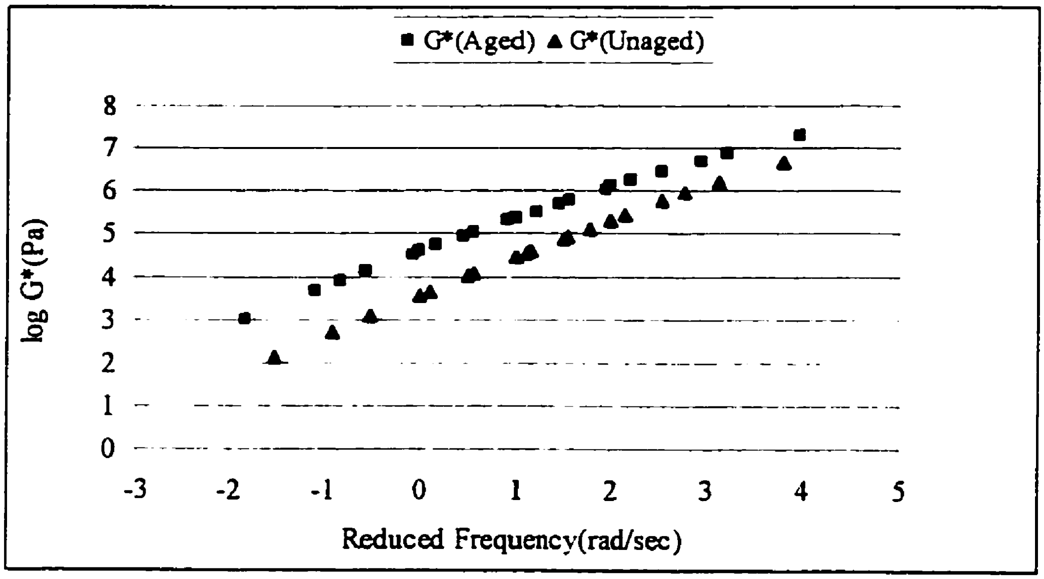


Figure 528 Master Curves for Unaged and Aged of C3 Blended Binder

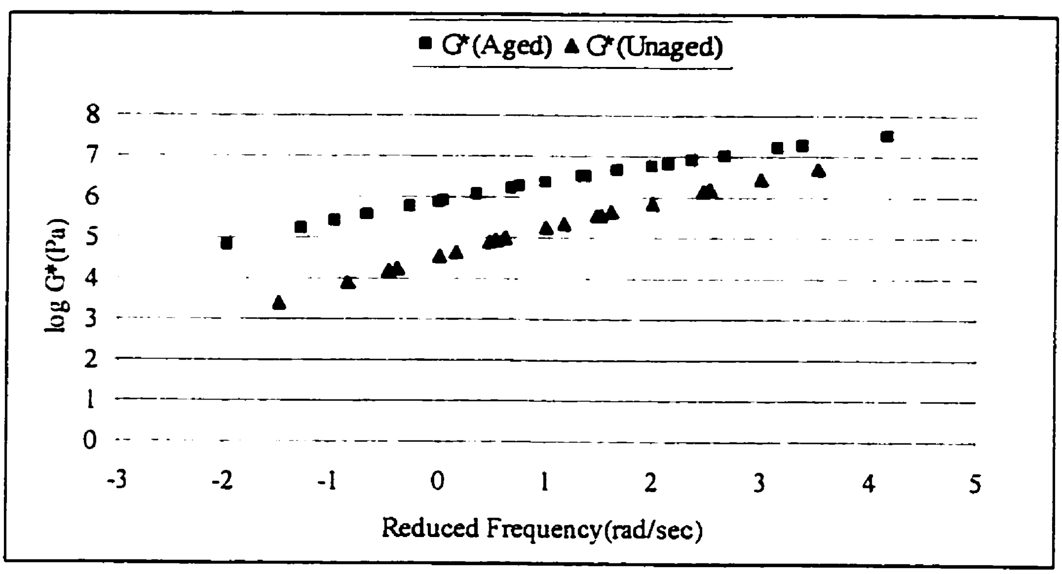


Figure 5. 29 Master Curves for Unaged and Aged of F1 Blended Binder

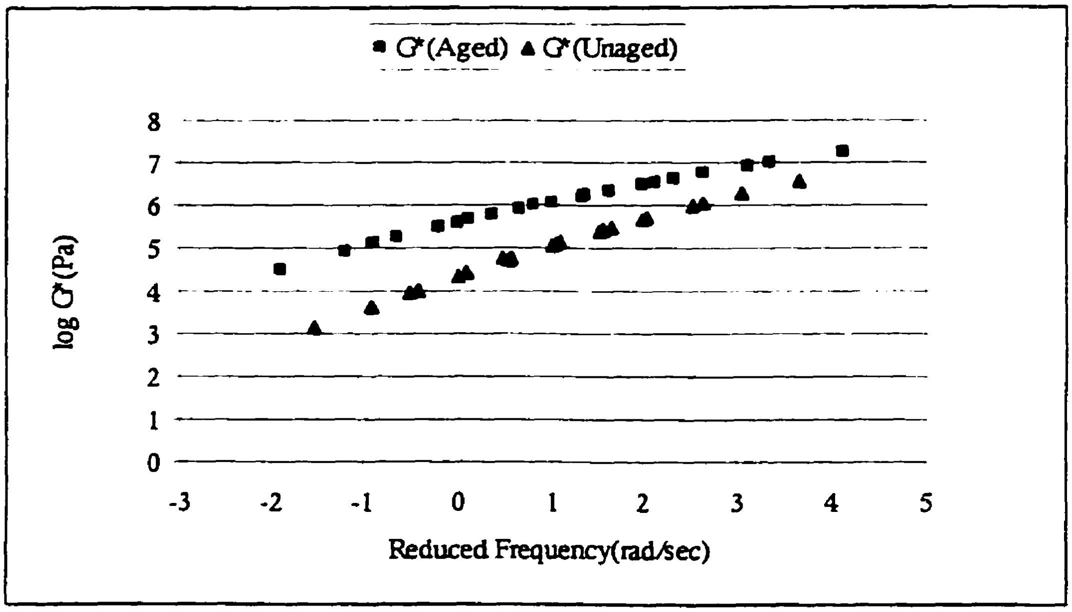


Figure 5.30 Master Curves for Unaged and Aged of F2 Blended Binder

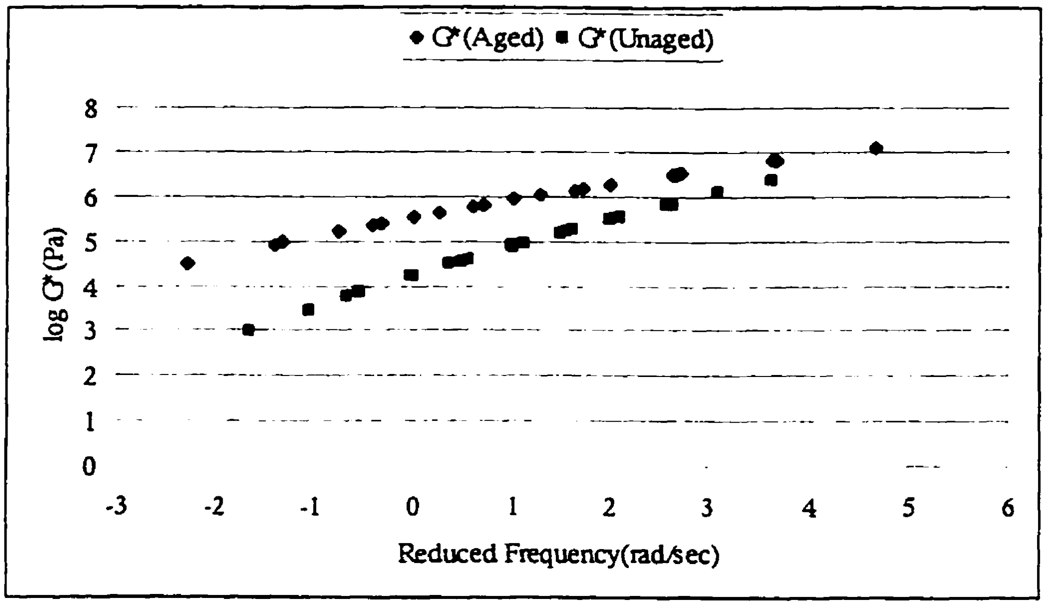


Figure 5. 31 Master Curves for Unaged and Aged of F3 Blended Binder

Figures 5.32 to 5.34 compare the master curves for different unaged and aged blended binders. Comparing master curves of unaged binders blended with soft asphalts, i.e. M1, M2, N1, and N2, the N1 binder had the highest modulus at low frequencies (high temperatures), but this group of binders behaves similar to each other at high frequencies (low temperatures). Master curves of N2 and M2 blended binders showed very close modulus at all frequencies.

For unaged binders blended with Cyclogen, the modulus decreased as the proportion of recycling agents in the blend increased. For binders blended with Cyclogen, the difference among modulus was very high at low frequencies (high temperatures), but it decreased at high frequencies (low temperatures). The shape of master curves for binders blended with Cyclogen is similar before and after aging, but shape of master curves for binders blended with Flexon changed after aging. This indicates the important effect of aging for binders blended with Flexon.

5.5 CHANGE IN R AND ω_C OF BLENDED BINDERS

The relationship between rheological parameters (R and ω_c) and proportion of recycling agents (percentage of weight) was studied. Linear and non-linear models were compared to study the goodness of fit for the linear model. The statistical analysis for linear and non-linear models is presented in Table 5.8. The statistical analysis strongly suggests that a linear relationship could explain the change in rheological parameters versus proportion of recycling agent in blend. The lowest correlation for both parameters are related to binders blended with Flexon. This is in agreement with the results of DSR and BBR for the same binders discussed in Chapter Four.

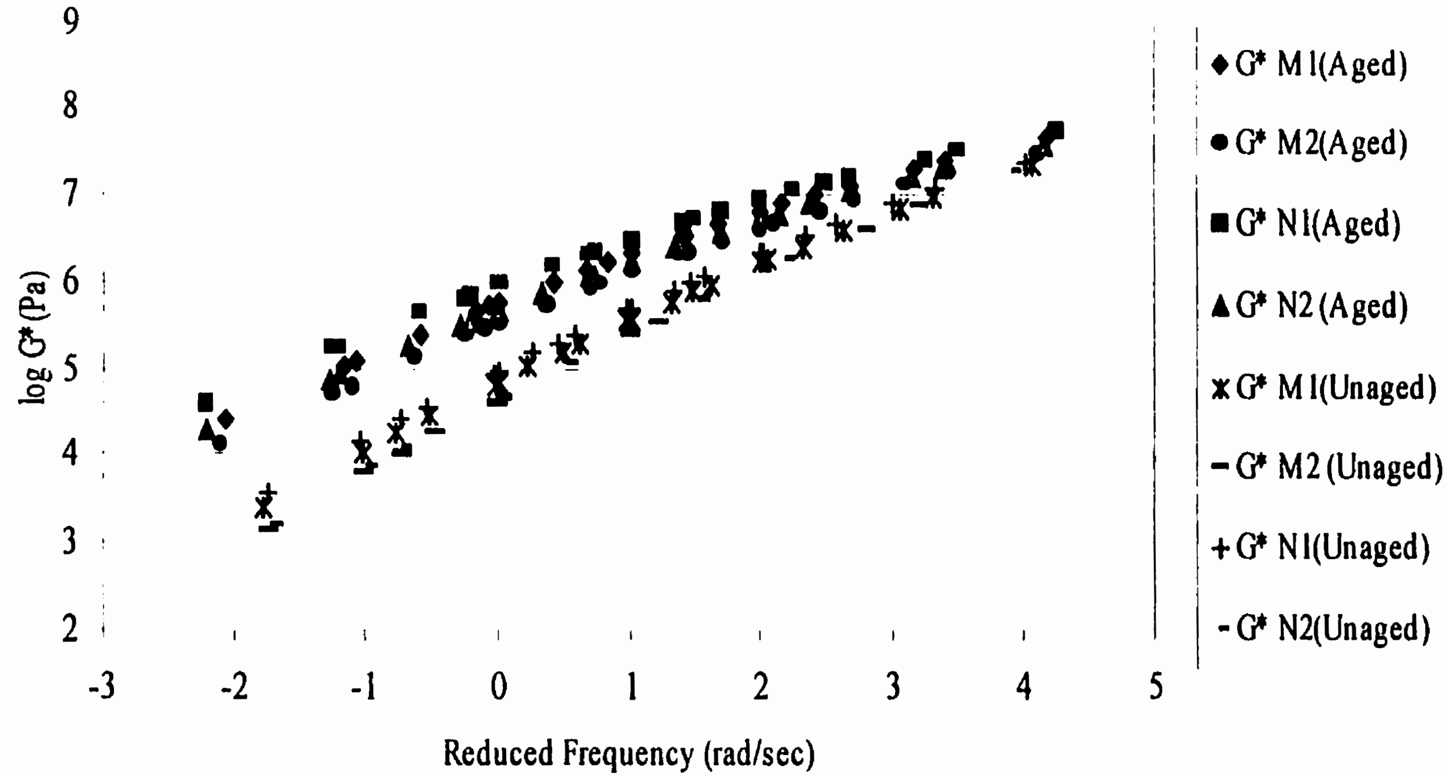


Figure 5.32 Master Curves for Binders Blended with 200-300 and 300-400 Asphalt Cement

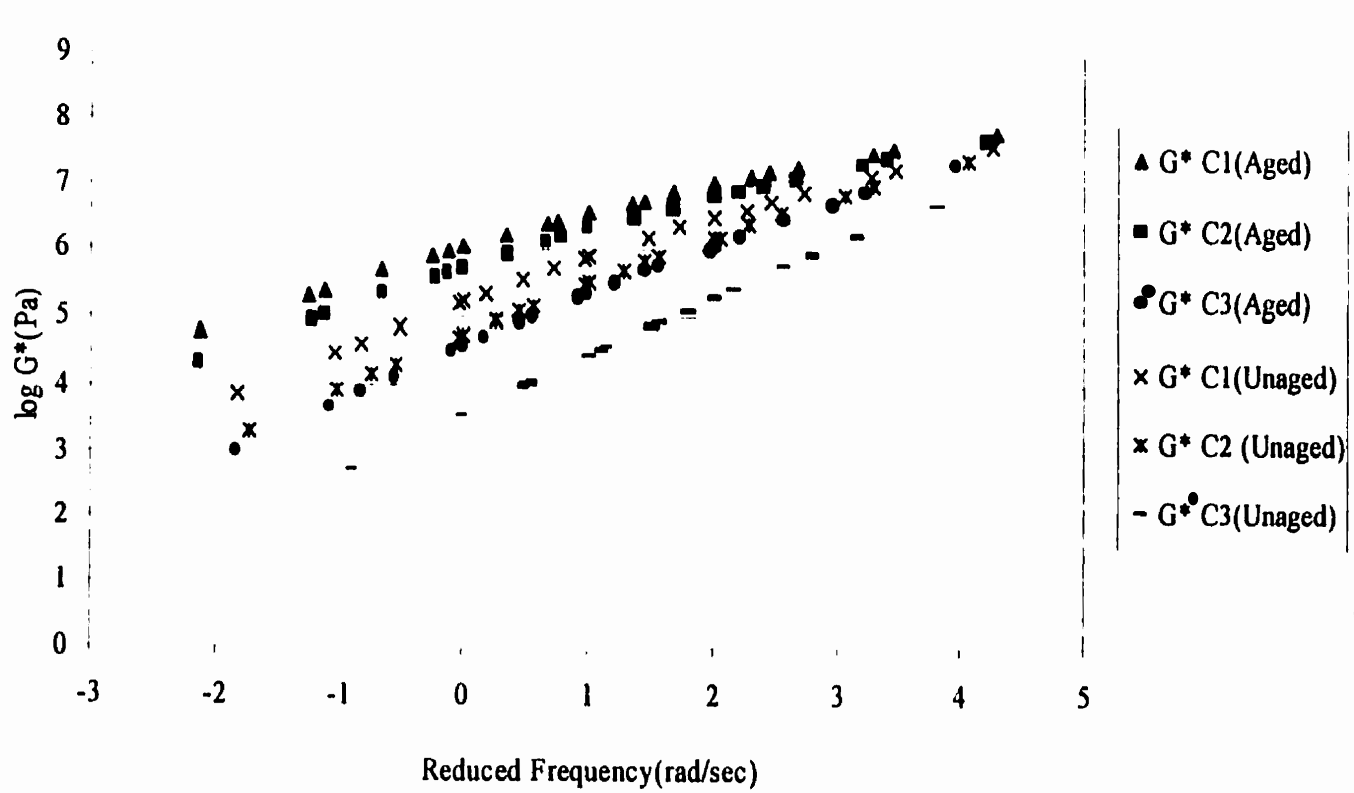


Figure 5.33 Master Curves for Binders Blended with Cyclogen

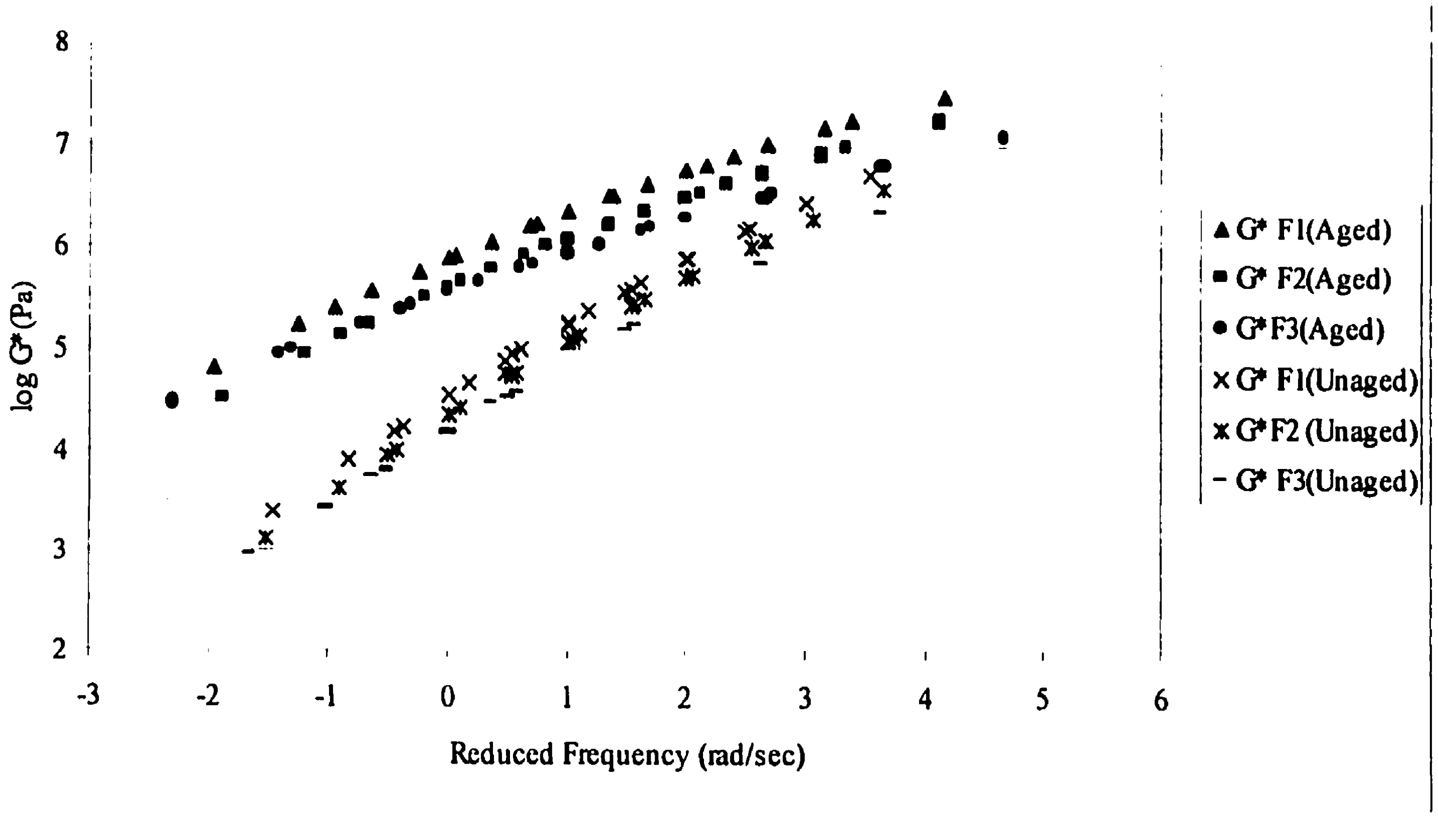


Figure 5.34 Master Curve for Aged Binders Blended with Flexon

Table 5. 8 Evaluation Criteria for Linear and Non-Linear Master Curve Parameters for Binders Blended

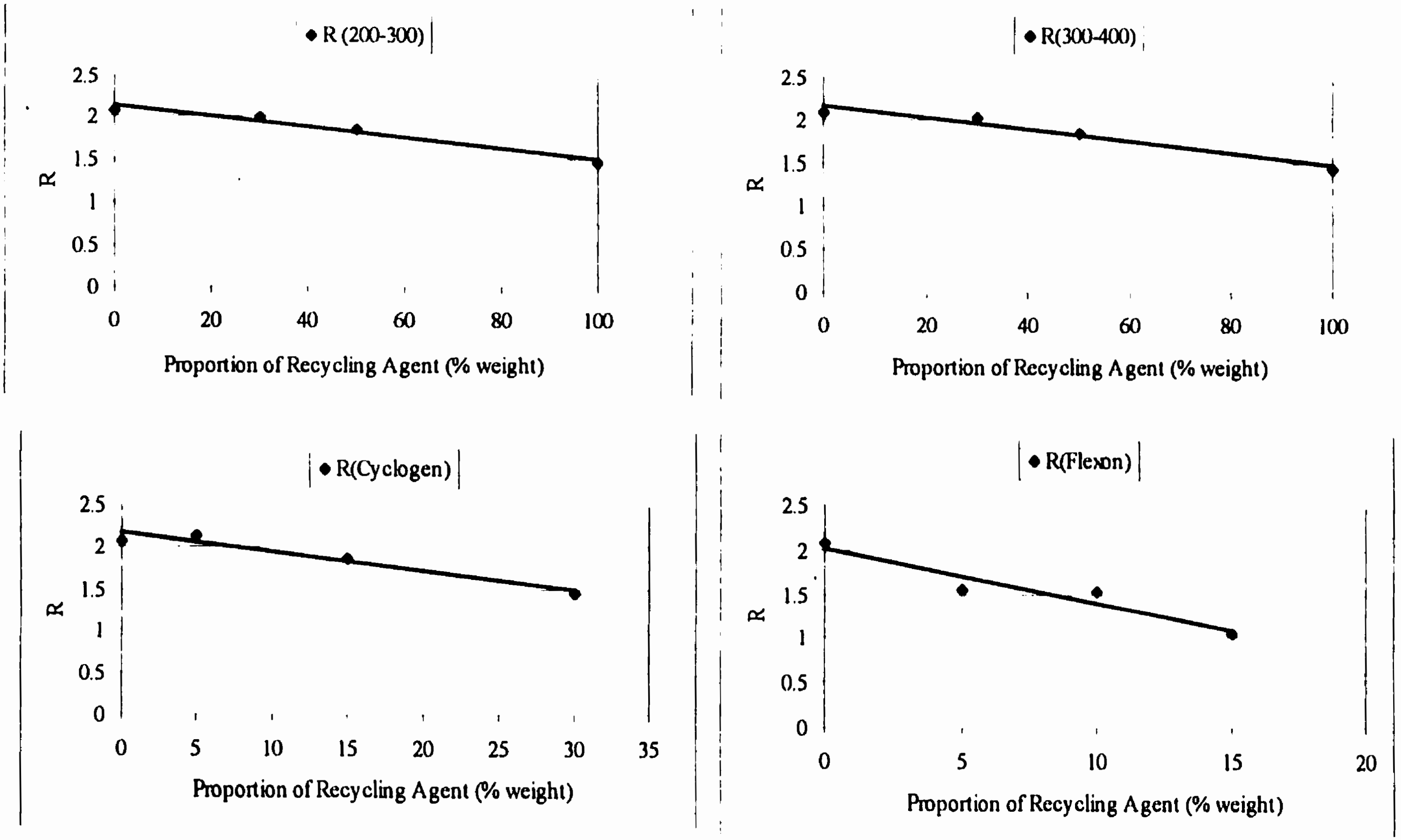
Parameters	Rheological Index (R)				Crossover Frequency (ω_c)			
	R^2		MSE		R^2		MSE	
Recycling Agent In Blend	Linear	Non-Linear	Linear	Non-Linear	Linear	Non-Linear	Linear	Non-Linear
200-300	0.9678	0.9964	0.0600	0.02816	0.9961	0.9995	0.0778	0.0232
300-400	0.9489	0.9911	0.0810	0.0478	0.9969	0.9981	0.0767	0.0850
Cyclogen	0.9368	0.9743	0.0945	0.0853	0.9908	0.9992	0.1518	0.06180
Flexon	0.9112	0.9113	0.1547	0.2187	0.9526	0.9943	0.2109	0.1028

Figures 5.35 to 5.42 show the change in R and ω_c with proportion of recycling agent in the blends. As recycling agents (percentage of mass) in the blends increased the R decreased and ω_c increased. This means that the binders are becoming softer with increasing recycling agent in the blends, which is expected.

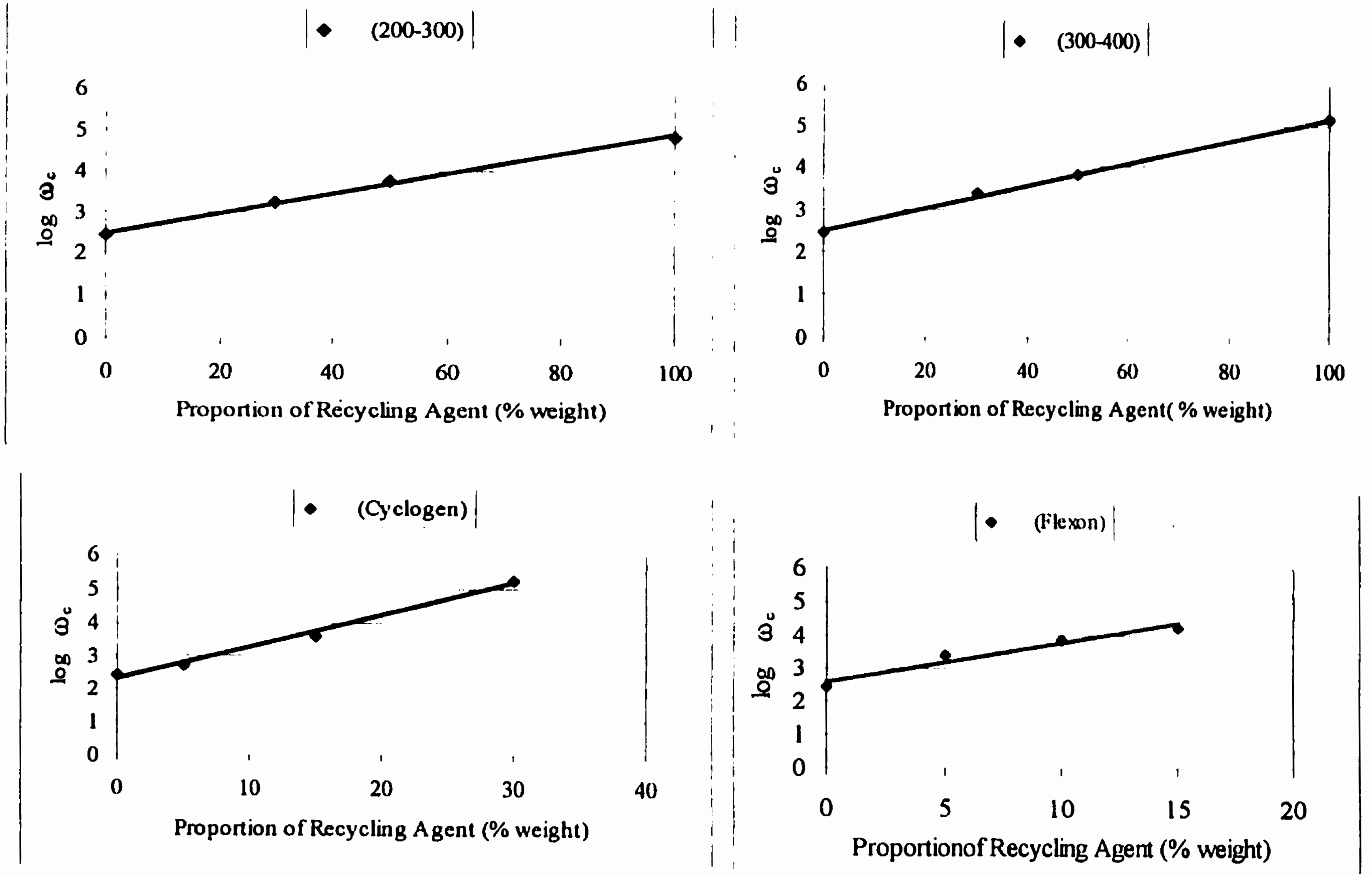
5.5 SUMMARY

The frequency dependency of complex shear modulus for blended binders was studied using master curves. For this purpose, the complex shear moduli for various frequencies were calculated, with SHRP A-002A model, using laboratory measured of 10 rad/s. The time-temperature superposition concept was used for calculating shift factors. A FORTRAN computer program was used to shift all frequency sweeps data on the 25°C line and shift factors of blended binders were calculated. The master curves of all blended binders were built.

The SHRP A-002A model was used, as the latest model, to study the frequency dependency of complex shear modulus. The rheological index (R) and crossover frequency (ω_c) of blended binders were estimated with the SPSS Program. Analysis of results showed that characterization of binders with master curves is a powerful tool. The estimated Rs could distinguish the binders based on the energy storage of asphalt cement during deformation. This means binders with higher R-values will store more energy during deformation than asphalts with lower R-values. The other estimated parameter, crossover frequency (ω_c), is an indicator of the overall hardness of a given asphalt. The lower the crossover frequency at a given temperature, the harder will be the asphalt. Therefore, binders with higher R and lower ω_c will be harder and more susceptible for cracking at low temperatures. Comparison among binders with master curves was in agreement with comparison with PG performance criteria in Chapter Four.



Figures 5.35, 36, 37, and 38 Change in Rheological Index (R) with proportion of Recycling Agent (% mass) in the Blend



Figures 5.39, 40, 41, 42 Change in Crossover Frequency (ω_c) with Proportion of Recycling Agent (% mass) in the Blends

he temperate dependency of shift factors was studied with the parameter T_d, defining temperature from WLF equation. The relationships of R, ω_c and T_d with proportion of recycling agent in the blends were studied. As the recycling agent in the blend increased, R decreased and ω_c increased. The conclusion was that a linear relationship is accurate enough for prediction of R and ω_c with percentage of weight of recycling agent in the blends.

CHAPTER SIX

CONCLUSIONS AND RECOMMENDATIONS

6.1 SUMMARY

This research program had two main objectives. The first objective was to characterize blended binders with the SHRP PG binder system and determine if a linear relationship exists between PG parameters and the proportion of recycling agent in blends. One virgin binder aged artificially in the lab was blended with two soft asphalt cements and two recycling agents, to create ten different blends. These blends were characterized at a range of pavement temperatures between –30° to 70°C with the Dynamic Shear Rheometer (DSR) and the Bending Beam Rheometer (BBR).

The other objective of this research was to characterize the blended binders with master curves for loading time dependency of blended binders. The relationship between master curve parameters (R and ω_c) and proportions of recycling agents were studied. Other relevant subjects also studied were:

1. The repeatability of DSR and BBR testing results
2. Temperature dependency of G*, δ, S, and m-value.
3. The loading time dependency for G* of blended binders.
4. New methods for the selection of recycling agent.
5. A case study for comparing the predicted performance of different recycling agent for Saskatchewan climatic conditions.

The two broad objectives for this research have been met. Several conclusions can be derived from this research. Also some recommendations useful for future studies can be made.

6.2 CONCLUSIONS

Based on the literature review, testing, and data analysis in this research the following conclusions can be drawn:

1. The current design methods for the selection of recycling agents are based on penetration and/or viscosity tests. These methods do not characterize blended binders across the range of pavement temperatures and loading times found in the field. The PG binder system has several advantages for characterization of asphalt cements and blended binders. As a linear viscoelastic material, asphalt cements or blended binders can be characterized with the PG binder system at a wide range of temperatures and loading times.

2. Generally, the coefficient of variations of PG testing parameters, from this study and other studies, are very high. Therefore, the PG system needs modification on the testing procedures and equipment. The precision of testing results was in an acceptable range when compared to other PG repeatability studies. The coefficient of variations of complex shear modulus was less than 10 percent more than other studies but the coefficient of variations of phase angle, stiffness

and m-value were in the same range of other studies. The precision of the Bending Beam Rheometer was generally better than the Dynamic Shear Rheometer.

3. A linear relationship was shown to be adequate for the prediction of PG testing parameters (log G*, δ, log S, and m-value) and performance criteria parameters (log G*/sinδ, log G*.sinδ, log S and m-value) with the proportion of recycling agents by weight in the blends.

4. Two methods were proposed for the selection of recycling agents in recycled mixtures. The first method was based on a linear relationship for PG binder performance criteria (G*/sinδ, G*.sinδ, S and m-value). The second method was based on a linear relationship between the temperatures that the PG criteria were satisfied with proportion of recycling agent.

5. The temperature dependency of blended lines for log G* and log S was studied. The statistical analysis showed that a shift value could be used for prediction of complex shear modulus and stiffness at other temperatures than the measured temperature. The temperature dependency of blended lines for phase angle and m-value also was studied and a logarithmic model was selected as the best model for this purpose.

6. Different blended binders were compared for a recycling project in Saskatchewan climatic conditions, as a typical cold climate Canada. The rutting and fatigue PG criteria could be meet using soft asphalt like 300-400. However, the same binder could not satisfy the low-temperature PG criteria with even 50% reliability. This is not in agreement with other studies for the same situation. This shows the PG performance prediction parameters or grading procedure need modifications for cold climate conditions.

dependency of blended binders. A computer program and non-linear regression analysis were used to build the master curve and to calculate the shift factors (a_T). The master curve was able to typify the blended binders at a wide range of frequencies.

8. The relationship of master curve parameters (R and ω_c) of the blended binders was studied. A linear relationship was accurate enough for prediction of these parameters of blended binders with proportion of recycling agents in the blends.

9. The temperature dependency of shift factors was studied based on defining temperature T_d from WLF Equation. The defining temperature decreased as the proportion of recycling agent in the blend decreased and the binders aged.

6.3 RECOMMENDATIONS

Recommendations are made for future studies in this area, as follows:

1. This study considered only a limited number of asphalt cements and recycling agent samples. The results need to be validated with additional binder samples, including binders extracted from RAP.

2. It is necessary to study the relationship between artificially laboratory aged and recovered binders from different Reclaimed Asphalt Pavement (RAP).

3. Master curve parameters (R and ω_c) were found useful characteristic parameters for asphalt cements. More research is needed to correlate these parameters with asphalt mixture performance. With having some criteria based on these parameters, it is possible to have better performance-based binder system in the future.

4. The interaction parameter, G_{12}, in Irving Equation is suggested to be studied for better correlation when a very soft recycling agent is blended with aged asphalt cement.

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APPENDIX A

PG TESTING RESULTS

Table A.1 and A.2 The Dynamic Shear Rheometer (DSR) Results for Unaged Binders and Blended Binders

	150-200		200-300		300-400	
Temp. °C	**G* Pa**	**δ degree**	**G* Pa**	**δ degree**	**G* Pa**	**δ degree**
7	4209600	59.75	1978900	66.20	1021200	68.50
13	1370860	66.20	620910	71.35	306700	73.00
19	463160	70.70	196800	74.30	101560	76.00
25	166670	73.90	66030	77.35	38290	79.30
31	61470	77.13	22850	80.15	13200	81.75
37	22900	80.52	9500	82.70	4480	84.03
46	5200	83.68	3060	85.10	1660	86.40
52	2250	85.58	1390	86.80	800	87.80
58	1060	86.92	600	88.17	400	88.90
64	590	88.00	310	89.30	200	89.35
70	300	88.85	160	89.77	115	89.60

	N1		N2		M1		M2	
Temp. °C	**G* Pa**	**δ degree**	**G* pa**	**δ degree**	**G* Pa**	**δ degree**	**G* Pa**	**δ degree**
7	7770100	46.6	4960100	55.5	6543000	47.6	4002800	53.9
13	3334450	52.1	1784200	59.5	2531675	53.1	1701050	59.0
19	1152350	57.1	645930	64.0	948050	58.0	597510	63.2
25	486850	61.1	268650	66.3	364560	62.1	249860	66.7
31	203530	65.6	110630	68.6	156465	65.6	109250	69.5
37	88520	69.7	47360	71.8	66000	69.7	40900	72.9
46	25050	75	11830	76.2	16980	75.0	10190	77.0
52	11140	78.2	5300	79.0	7270	78.2	4260	80.1
58	5110	81	2590	81.7	3290	81.0	2010	82.8
64	2430	83.8	1180	84.3	1540	83.8	988	84.5
70	1210	85.6	630	86.0	747	85.6	510	87.0

Table A.3 and A.4 The Dynamic Shear Rheometer (DSR) Results for Aged Binders

	150-200		200-300		300-400	
Temp. °C	**G* Pa**	**δ degree**	**G* pa**	**δ degree**	**G* Pa**	**δ degree**
7	18274380	39.1	6531950	46.6	4821300	48.5
13	8798750	45.5	2506400	51.9	1850650	53.5
19	3212160	50.9	1014965	56.2	750370	57.8
25	1369060	55.5	432880	60.4	311300	61.3
31	583450	59.0	173270	63.9	126090	64.9
37	256600	62.3	68780	67.1	52410	68.1
46	58860	67.2	19063	72.0	13950	73.4
52	25030	71.4	8490	76.1	5990	77.2
58	11320	75.6	3885	78.9	2750	80.2
64	5190	78.5	1746	81.6	1250	83.0
70	2470	81.1	855	84.5	620	85.3

	N1		N2		M1		M2	
Temp. °C	**G* Pa**	**δ degree**	**G* Pa**	**δ degree**	**G* Pa**	**δ degree**	**G* Pa**	**δ degree**
7	24739000	31.6	15366500	34.5	19282000	34.0	11899800	36.0
13	13087000	36.3	7502800	39.0	9571000	38.4	6245750	40.5
19	6099950	40.0	3656750	43.1	4489750	43.0	2855650	44.6
25	2938250	43.5	1674650	46.6	2072750	46.9	1226050	48.5
31	1487000	46.5	724250	50.3	1001950	50.5	541810	52.2
37	646000	50.1	321500	53.1	460820	54.2	231850	55.7
46	174050	54.3	83990	56.4	124150	59.0	65370	60.0
52	81600	58.0	38350	60.4	53900	62.7	27175	63.9
58	40330	61.2	18510	64.3	24520	67.0	14315	67.1
64	20550	65.1	9200	68.7	12250	71.5	7615	72.0
70	9610	67.7	4500	71.8	6060	75.3	3830	75.2

Table A.5 and A.6 The Dynamic Shear Rheometer (DSR) Results for Blended Binders (Unaged) With Cyclogen and Flexon

	C1		C2		C3	
Temp. °C	**G* Pa**	**δ degree**	**G* Pa**	**δ degree**	**G* Pa**	**δ degree**
7	13530000	43.8	6788800	53.1	876195	70.8
13	5603750	48.3	2384700	58.4	273705	73.5
19	2200300	52.4	807795	62.2	88030	76.2
25	801995	56.8	329910	65	29020	79.2
31	368210	60.3	127965	68.3	10250	81.7
37	161900	63.4	54680	71.1	4540	83.5
46	42960	68	14560	75.3	1210	86.5
52	16540	71.1	5425	78.6	560	87.9
58	7630	74.6	1990	81.5	270	88.7
64	3720	77.2	940	83.8	145	89.3
70	1720	80.2	450	85.6	70	89.8

	F1		F2		F3	
Temp. °C	**G* Pa**	**δ degree**	**G* Pa**	**δ degree**	**G* Pa**	**δ degree**
7	1499950	51.3	1157600	51.7	665010	54.2
13	745430	54.8	519810	56.5	341445	58.2
19	354880	57.2	249970	58.9	163560	61.3
25	173220	59.8	111645	61.2	73370	63.4
31	94950	62.6	56430	64.4	33970	66.6
37	46260	65.4	26090	67.5	15270	70.5
46	15910	69.4	9210	72.8	5560	74.5
52	7880	72.8	4850	76.7	2785	78.1
58	3940	75.8	2410	79.7	1340	82.7
64	1930	79	1165	82.7	660	84.2
70	958	81.8	540	85.0	320	86.6

Table A.7 and A.8 The Dynamic Shear Rheometer (DSR) Results for Blended Binders (Aged) With Cyclogen and Flexon

	C1		C2		C3	
Temp. °C	**G* Pa**	**δ degree**	**G* Pa**	**δ degree**	**G* Pa**	**δ degree**
7	30279500	30.5	20324000	35.6	4980250	58.0
13	15103000	35.1	9628850	40.2	1708100	61.3
19	7366500	39.3	4434350	44.2	600255	64.7
25	3628450	42.8	2029750	47.8	232750	68.1
31	1715450	46.5	901120	51.3	87160	71.2
37	822610	49.6	405645	54.7	32475	75.0
46	252620	53.5	106050	58.9	7620	79.0
52	102130	56.8	45680	62.1	2990	82.9
58	50130	59.8	21260	66.3	1140	85.2
64	24500	63.4	9925	70.2	480	86.6
70	14020	66.1	4660	74.3	235	88.0

	F1		F2		F3	
Temp. °C	**G* Pa**	**δ degree**	**G* Pa**	**δ degree**	**G* Pa**	**δ degree**
7	15075500	30.1	8236450	51.7	6370850	54.2
13	8121300	33.6	4234700	56.5	3098600	58.2
19	4291950	36.9	2206850	58.9	1529900	61.3
25	2216150	39.9	1141000	61.2	840500	63.4
31	1118350	42.7	568910	64.4	437290	66.6
37	558400	45.5	299720	67.5	230890	70.5
46	238600	49.5	125920	72.8	91875	74.5
52	131960	53.1	56670	76.7	42500	78.1
58	55010	56.6	27630	79.7	19510	82.7
64	28530	59.7	14000	82.5	9720	84.2
70	12460	63.8	6850	85.0	4980	86.6

Table A.9 The Dynamic Shear Rheometer (DSR) Results for Once Aged and Double Aged of 150-200 Binder

	150-200 (RUN1)*		150-200 (RUN2)**	
Temp °C	**G* (Pa)**	**δ (degree)**	**G* δ**	**(Pa) (degree)**
7	18274380	39.1	30132000	29.8
19	3212160	50.9	7849000	38.9
31	583450	59.0	1898000	45.7

* RUN 1 is the first aging of 150-200 in the RTFO and PAV.
** RUN 2 used the material from RUN 1 to aged in the RTFO and PAV.

Table A.10, A.11 The Bending Beam Rheometer (BBR) Results for PAV Asphalt Binders and Binders Blended with 200-300 and 300-400

Binders	**150-200 Aged**		**150-200 (Double Aged)**		**200-300**		**300-400**	
Temp. °C	**m**	**S(MPa)**	**m**	**S(MPa)**	**m**	**S(MPa)**	**m**	**S(MPa)**
-18	0.352	55.57	0.311	225	0.441	84.32	0.452	48.62
-24	0.303	340.60	0.268	434	0.378	216.04	0.387	142.83
-30	0.216	555.00	0.212	781	0.302	423.80	0.316	337.45

Binders	**M1**		**M2**		**N1**		**N2**	
Temp. °C	**m**	**S(MPa)**	**m**	**S(MPa)**	**m**	**S(MPa)**	**m**	**S(MPa)**
-18	0.358	136.5	0.381	124.8	0.333	176.3	0.357	141.1
-24	0.315	290.7	0.339	268.5	0.273	397.95	0.299	329.6
-30	0.227	590.8	0.246	543.9	0.229	712.15	0.249	597.7

Tables A.12, and A.13 The Bending Beam Rheometer (BBR) Results for PAV Binders Blended with Cyclogen and Flexon

Binders	C1		C2		C3	
Temp. °C	S(MPa)	m	S(MPa)	m	S(MPa)	m
-30	732.87	0.222	685.93	0.247	590.85	0.294
-24	405.95	0.272	359.85	0.310	246.90	0.412
-18	204.43	0.325	162.80	0.375	120.25	0.425

Binders	F1		F2		F3	
Temp. °C	S(MPa)	m	S(MPa)	m	S(MPa)	m
-30	370.58	0.256	144.50	0.289	114.60	0.303
-24	154.30	0.297	77.36	0.301	66.13	0.317
-18	81.44	0.335	47.11	0.345	37.59	0.353

APPENDIX B

COMPUTER PROGRAM FOR CALCULATING SHIFT FACTORS OF BLENDED BINDERS

```
cccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccc
C                         SHIFT COMPUTR PROGRAM
c This computer program calcultes shift factors. The calculatd shift
c factors are used to build mater curves for asphalt cements.
c This program uses Newton-Raphson method to calculate shift factors.
ccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccc

          integer i,m
          parameter(n=9,m=7,k1=7,k2=k1*3)
          double precision w(n),g(m,n),t(m),atavm(k1),gv(k2),lg109(k2)
          &,wa(k2),wal(k2),atm(k1,m)
          double precision h0,rm,wcm,wm,gm,dgdwm,g1m,xm
          character temp*3,title*30
          external derivs

ccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccc
c Read input data (complex shear modulus g(m,n) at different tempertures
c and frequencies w(n)) from a file.
c write input data to a file.
ccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccc
          open(unit=11, file='hamid1.txt',status= 'old')
          read(11,*) title
          read(11,*) rm,wcm
          read(11,*) temp,(w(j),j=1,n)
          do 100 i=1,m
           read(11,*) t(i),(g(i,j),j=1,n)
100         continue
         close(unit=11)

          open(unit=12, file='hamid1.out',status= 'unknown')
          write(12,'(a30)') title
          write(12,*) rm,wcm
          write(12,'(a5,9f15.2)') '    t',(w(j),j=1,n)
          do 110 i=1,m
           write(12,'(f5.2,9f15.2)') t(i),(g(i,j),j=1,n)
110           continue
          write(12,'(80a)') ('-',j=1,80)
          write(12,'(a5,9f15.2)') '    t',(w(j),j=1,n)
          write(12,'(80a)') ('-',j=1,80)

          do 1202 j=1,n
           w(j)=10.0**w(j)
1202          continue
          write(12,'(a5,1p9e15.2)') '    t',(w(j),j=1,n)
          write(12,'(80a)') ('-',j=1,80)
```

```
ccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccc
c This part of program, calculates the shift factors for each complex shear
c modulus and calculates the mean of shift factors for each temperature.
c Roots of isothermal curves are calculated by Newton-Raphson method in
c subrotuine 'geqg1'.
ccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccc
          do 1000 k=1,k1
           write(12,'(a,t10,i3)') 'row=',k
             summ=0.0
             do 1200 j=5,7
            wm=log10(w(j))
              g1m=log10(g(k,j))
            write(12,'(8a10)') 'r','wc','w','g','dgdw','g1'
            write(12,'(8e10.2)') rm,wcm,wm,gm,dgdwm,g1m
            write(12,'(8a10)') 'xhj','g','dgdw','mhj','dmhj','hj1'
              call  geqg1(h0,rm,wcm,wm,gm,dgdwm,g1m,xm)
              atm(k,j)=xm
              summ=summ+atm(k,j)
            write(12,'(40a)') ('-',j2=1,40)
1200         continue
             atavm(k)=summ/3.0
           write(12,'(4e15.6)') (atm(k,j),j=5,7),atavm(k)
           write(12,'(80a)') ('-',j=1,80)
1000         continue
             close(unit=12)

ccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccc
c This part of program, shifts all complex shear modulus into the base curve
c (25 dgree C) and writes the final output of the code into a file.
ccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccc
          open(unit=13, file='hamid2.out',status= 'unknown')
          write(13,'(a30)') title
          write(13,990) 'g*','log(at)','log(at)','log(g*/gg)','w'
          do 200 j=1,3
           do 2000 k=1,k1
              kj=(j-1)*k1+k
              j5=j+4
              gv(kj)=g(k,j5)
              lg109(kj)=log10(gv(kj)/1.0e9)
              if(k.ge.1.and.k.le.4) then
                wa(kj)=atavm(k)+j-1.0
                wal(kj)=10.0**wa(kj)
              end if
              if(k.ge.5.and.k.le.7) then
                wa(kj)=-atavm(k)+j-1.0
```

```
                wal(kj)=10.0**wa(kj)
              end if
            write(13,900) gv(kj),wa(kj),atavm(k),lg109(kj),wal(kj)
2000          continue
200           continue
900           format(f17.2,4f15.6)
990           format(t16,a,t26,a,t41,a,t53,a,t77,a)
9900          end

ccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccc
c Subroutine 'derivs' calculates complex shear modulus and its first
c derivative based on "SHRP A-002A" model to be used in subrotuine
c 'geqg1' for root finding.
ccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccc
         subroutine derivs(r,wc,w,g,dgdw,g1)
         double precision r,wc,w,g,dgdw,g1
         double precision a,gg,dg2dw2
         a=0.3010299957/r
         gg=1.0e9
         w=10.0**w
         if(abs(w).lt.1e-6) then
          write(*,'(a30)') 'w is approaching zero'
             goto 100
         end if
         g=(gg*(1.0+(wc/w)**a)**(-1.0/a))
         dgdw=gg/w*((wc/w)**a)*((1.0+(wc/w)**a)**((-a-1.0)/a))
         dg2dw2=w*dgdw/g
         dgdw=dg2dw2
         g=log10(g)-g1
         w=log10(w)
100           return
              end
ccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccc
c Subroutine 'geqg1' calculates the root of the base complex shear
c modulus curve 'g' (25 dgree C) for any given complex shear modulus
c (line 'g1') using Newton-Raphson method.
ccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccccc
         subroutine geqg1(h0,r,wc,w,g,dgdw,g1,x)
         integer i
         double precision h0,r,wc,w,g,dgdw,g1,x
         double precision xj,hj,xhj,mhj,dmhj,hj1,xs
         external derivs

         i=1
         xj=w
```

```
      hj=0.0
      xs=xj
150      xhj=xj+hj
      call derivs(r,wc,xhj,g,dgdw,g1)
      mhj=g
      dmhj=dgdw
      hj1=-mhj/dmhj
      write(12,'(8e10.2)') xhj,g,dgdw,mhj,dmhj,hj1
      if(max(abs(mhj),abs(hj1)).lt.1.0e-3) then
        xs=xhj
        go to 170
      end if
      xj=xhj
      hj=hj1
      i=i+1
      if(i.gt.20) goto 170
      go to 150
170   x=abs(xs-w)
      write(12,'(8a10)') 'xs','w','x'
      write(12,'(8e10.2)') xs,w,x
      return
      end
```

A Typical Input Data for SHIFT Computer Program for Unaged 150-200 Asphalt Cement

1)	'v150-200'								
2)	1.5215355	22153.88							
3) 'T'	-4	-3	-2	-1	0	1	2	3	4
4) 7	0	0	0	0	4209600.0	17167473.81	54048461.51	0	0
5) 13	0	0	0	0	1370860.0	6671937.29	25392909.68	0	0
6) 19	0	0	0	0	463160.0	2564599.09	11359984.46	0	0
7) 25	37.89	343.44	2952.50	23467.28	166670.0	1014406.70	5059280.65	19897573.64	60434241.79
8) 31	0	0	0	0	61470.0	411318.82	2308048.61	0	0
9) 37	0	0	0	0	22900.0	169240.33	1075397.67	0	0
10) 46	0	0	0	0	5200.0	42348.15	307364.42	0	0

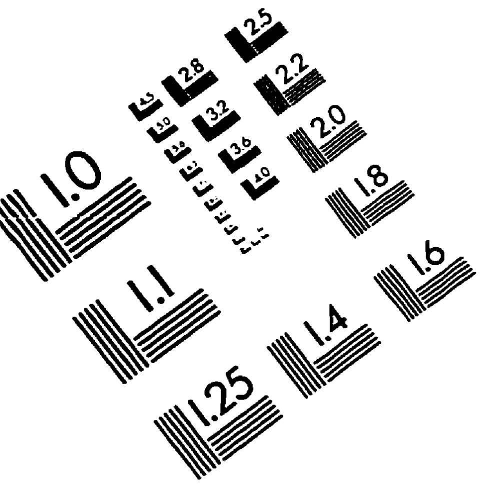
Line 2 R and ω_c for 25°C
Line 3 Log of frequency from –4 to +4 rad/s.
Line 4 to 10 Complx shear modulus at different temperatures (Pa)

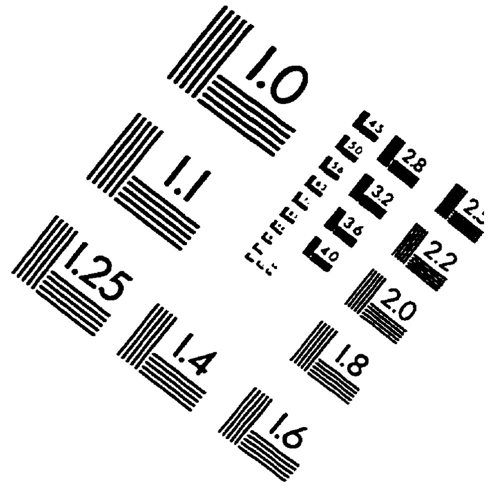
A Typical Output Data for SHIFT Computer Program for Unaged 150-200 Asphalt Cement

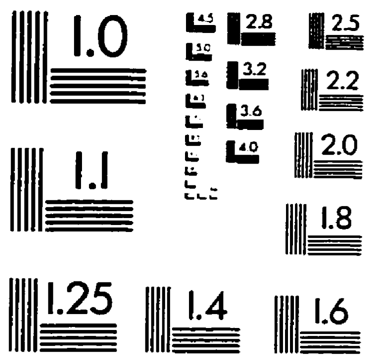
v150-200

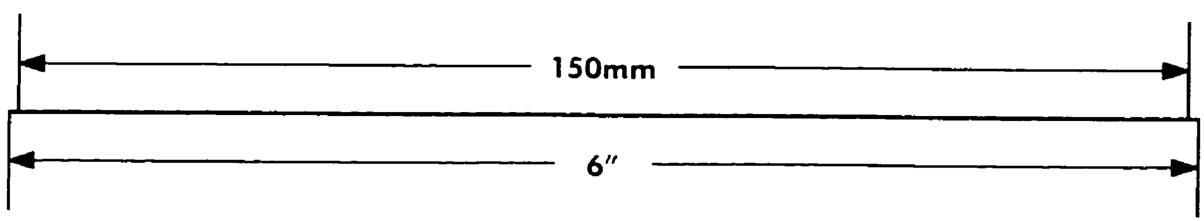
G*	log(at)	log(at)	log(G*/Gg)	ω_c
4209600.00	2.882479	2.882479	-2.375759	762.920056
1370860.00	2.188540	2.188540	-2.863007	154.361796
463160.00	1.560074	1.560074	-3.334269	36.313993
166670.00	.999676	.999676	-3.778143	9.992548
61470.00	-.488243	.488243	-4.211337	.324906
22900.00	-.018019	.018019	-4.640165	.959359
5200.00	-.703046	.703046	-5.283997	.198132
17167473.81	3.882479	2.882479	-1.765294	7629.200558
6671937.29	3.188540	2.188540	-2.175748	1543.617960
2564599.10	2.560074	1.560074	-2.590981	363.139935
1014406.70	1.999676	.999676	-2.993788	99.925476
411318.82	.511757	.488243	-3.385821	3.249058
169240.34	.981981	.018019	-3.771496	9.593587
42348.16	.296954	.703046	-4.373165	1.981318
54048461.51	4.882479	2.882479	-1.267217	76292.005582
25392909.68	4.188540	2.188540	-1.595288	15436.179598
11359984.46	3.560074	1.560074	-1.944622	3631.399350
5059280.65	2.999676	.999676	-2.295911	999.254764
2308048.62	1.511757	.488243	-2.636755	32.490576
1075397.68	1.981981	.018019	-2.968431	95.935870
307364.42	1.296954	.703046	-3.512346	19.813182

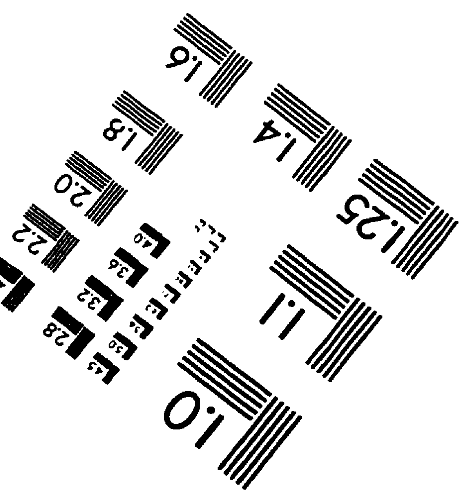
IMAGE EVALUATION TEST TARGET (QA-3)

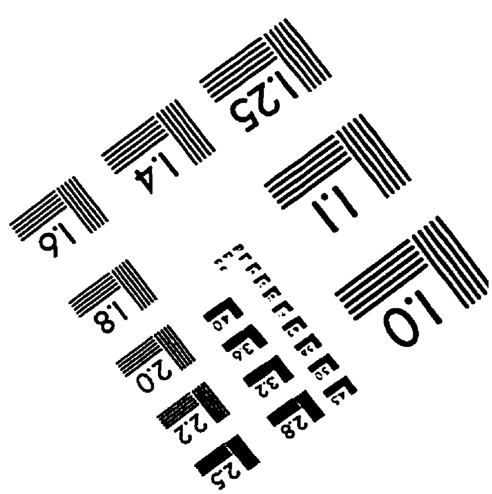












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